A MODEL TO PREDICT STRESS-DEFORMATION BEHAVIOUR OF NORMALLY CONSOLIDATED CLAY

A Thesis Submitted
in Partial Fulfilment of the Requirements
for the Degree of
MASTER OF TECHNOLOGY

By
V. KUGANATHAN

to the

DEPARTMENT OF CIVIL ENGINEERING
INDIAN INSTITUTE OF TECHNOLOGY KANPUR
AUGUST, 1976

To

MY PARENTS

CERTIFICATE

This is to certify that the thesis entitled 'A model to predict stress deformation behaviour of normally consolidated clays' by Mr. Kuganathan in partial fulfilment of the requirements for the degree of 'Master of Technology' at the Indian Institute of Technology, Kanpur is a record of bonafide research work carried out under my supervision and guidance. The work embodied in this thesis has not been submitted elsewhere for a degree.

YUDHBIR
Professor
Department of Civil Engineering
Indian Institute of Technology,
Kanpur.

August 2, 1976.

ACKNOWLEDGEMENTS

I have immense pleasure in acknowledging my deep sense of gratitute to my thesis supervisor Dr. Yudhbir, who has initiated me into the problem, and has with his guidance inspired me time and again.

I am extremely thankful to all the faculty members of the Civil Engineering Department, in particular to Dr. M.R. Madhav for making my stay very pleasent and enjoyable.

I am thankful to many of my colleagues and friends who have contributed directly or indirectly to this thesis. Special reference should be made of Mr. A. Arumugam whose companionship has helped me to complete this program successfully.

My thanks are also due to Mr. K.V. Lakshmidhar and other laboratory staff members for their admirable co-operation in the laboratory. Mr. Tewari's efficient typing is deeply appreciated.

V. KUGANATHAN

CONTENTS

-.-.-.-.-

NOTATIONS

A	stress path corresponds to conventional drained test
a	area of the sample
a_r	area of the piston
D'	Model parameter (constant)
e	void ratio
K	coefficient of lateral earth pressure
M	critical state frictional constant
p'	mean effective pressure
p_c	critical pressure
p_e	equivalent consolidation pressure (Hvorslev)
q	deviatoric stress $(\sigma_1 - \sigma_3)$
q_f	deviatoric stress at failure
α_1, α_3	model parameter (constants)
ε_v	volumetric strain
ε	distortional strain
η	anisotropic ratio q/p'
κ	slope of the swelling curve in e ln p plot
λ	slope of the consolidation curve in e ln p plot
ϕ	angle of internal friction.

SYNOPSIS

A MODEL TO PREDICT STRESS-DEFORMATION BEHAVIOUR OF NORMALLY CONSOLIDATED CLAYS

(A thesis submitted in partial fulfilment of the requirements for the Degree of Master of Technology by V. Kuganathan to the Department of Civil Engineering, Indian Institute of Technology, Kanpur, India.)

The stress deformation behaviour of normally consolidated wet clay along various stress paths in the plane of triaxial stresses has been the subject of investigation. Fully drained triaxial tests were carried out on specimens (11.68 cm^2 area) consolidated under 0.2 kg/cm^2 vertical pressure in perspex glass cylinders. The test program consisted of $p' = \frac{(\sigma_1' + 2\sigma_3')}{3}$ constant tests $q = (\sigma_1 - \sigma_3)$ constant tests, conventional drained and undrained tests on anisotropically consolidated specimens. The tests were designed to provide information on state boundary surface and the failure states of the clay.

The model parameter evaluation from the suitable plots of p'-constant tests and q-constant tests is explained and it is shown that in the practical range of η values these parameters are more or less constant. The prediction of conventional drained tests by the proposed model was

done and a very good agreement between the observed results and the prediction was obtained. Similarly it is shown that the proposed model predicts the anisotropic consolidation tests satisfactorily.

It is shown that the state boundary surface for drained tests and the state boundary surface for undrained tests are not the same. This contrasts with the hypothesis implicit in the Cambridge theory that the surface obtained from the shear tests on isotropically consolidated samples at different water contents can be used to predict the anisotropic behaviour of the clay.

CHAPTER 1

INTRODUCTION

1.1 PREDICTION IN GEOTECHNICAL ENGINEERING

According to Professor T.W. Lambe (1973), Predicting constitutes an integral component - the very heart- of the practice of Civil Engineering. The successful engineer must not only predict but ~~must~~ also take decisions and take actions on the basis of his predictions. Predicting is a Key step in the process of creating and maintaining a constructed facility. In addition to participating in prediction of construction costs, environmental impact and so on, the Geotechnical Engineer predicts many aspects of performance of Civil Engineering facilities. Some of the aspects of performance which the Engineer may be required to predict are

a. deformations (both magnitude and rate)

b. stability

c. Loads - both lateral and vertical applied to structures resting on or within soil. (e.g. struts, sheeting, pipes, tunnels etc.)

Although there are many techniques of predicting internal stresses, deformations and stability for a Geotechnical facility, the application of these techniques has limitations. The

major limitations are the difficulty of determining fully and accurately the field situation,the mechanisms which will occur, and the selection of soil parameters , to use with prediction methods.

1.2 STABILITY DEFORMATION BEHAVIOUR

Although most of the early research carried out on the behaviour of soils under the action of applied stresses was restricted to a study of peak strength conditions and failure criteria, an increasing amount of attention is being given recently to a study of pre-peak stress strain behaviour. This results from the realization that most foundation elements are stressed at levels significantly below the failure stress and a good prediction of deformations requires accurate determination of stress strain, behaviour in the range of working stresses.

Study of stress strain behaviour of soils with an objective to formulate simple and reliable predictive techniques was vigorously advocated by the Late Professor Roscoe (1970). Finite element method has been extensively employed to simulate actual field deformation conditions in the study of soil structure interaction problem. According to Prevost (1974) a point of diminishing returns has been reached with respect to the basic formulation of finite element method. Prevost stresses the need for the researchers to

devote much more attention to the development of realistic constitutive relations for soils. Main objective of this thesis is to attempt along this direction.

1.3 LABORATORY TESTING AND FIELD PREDICTIONS

It is always difficult to relate laboratory tests directly to field problems. We are offen faced not only with obvious non-homogeneity but also with the less obvious anisotropy which results from both microstratification and stress history.

The practice of Geotechnical Engineering requires that both non-homogeneity and anisotropy are handled in such a way that satisfactory engineering solutions are achieved in both the design and construction process.

In order to obtain these solutions we need some basic understanding of the way in which anisotropic materials behave and it is often more useful to carry out a few tests designed to throw light on the basic behaviour rather than produce a mass of test data of a routine nature.

1.4 STATEMENT OF THE PROBLEM

The scope of the present investigation is

(1) To consider how a sample which has been consolidated anisotropically to a particular stress state reacts to stress probing in any direction within the Triaxial stress plane.

(2) To verify the predictability of a model, which was proposed by Mathur (1976). This model is based on two simple Triaxial tests, viz.

$p' = \frac{\sigma_1' + 2\sigma_3'}{3}$ constant, and $q = \sigma_1 - \sigma_3$ constant.

CHAPTER 2

LITERATURE REVIEW

2.1 General - (before 1960).

In the extensive literature on the shear behaviour of soils much more attention has been concentrated upon investigation of the "strength properties" rather than upon the "deformation characteristics".

The case for a unique relationship between void ratio and effective stress for a saturated normally consolidated soil was first advanced by Rendulic (1937) who argued that the new void ratio after a change in effective stress is not dependent on the stress path but is exclusively determined by the three new principal effective stresses. His tests showed the relationship between drained and undrained tests under increasing stresses on isotropically consolidated sample. He further argued that tests on anisotropically consolidated samples would fall into the same pattern.

His work was later extended by Henkel (1958) who considered tests involving more elaborate stress paths.

Many attempts had been made to extend the theory of plasticity originally developed for metals, to model soil behaviour.

Predictions of shear strain is of considerable interest. Drucker et al (1957) considered that soil could be treated as conforming to the work hardening theories of plasticity and that a yield locus could be determined, or could establish the ratio of the vector components of the plastic strain increments.

Unfortunately the theories impose certain conditions on the behaviour of the media which are contradicted by experimental observations on real soils. Drucker summarized the state of conflict and implied that more complicated theories might be required to explain the observed behaviour.

The position in 1960 was well summarized by Henkel.

" So far it has not been possible to relate shear strains measured in the various types of tools. For a complete understanding of the stress strain behaviour of clays it is necessary that the shear stresses and shear strains should be related. Until this problem is solved it will not be possible to examine in any fundamental way the deformation behaviour of clays which have an important bearing on many practical problems".

Henkel was concerned only with triaxial test data, but his remarks are valid for all types of apparatuses.

(After 1960)

Calladine (1963) suggested a particular construction for the yield locus as forming a reasonable basis for

prediction of stress strain behaviour. The critical state concept and related application of plasticity theory extensively developed by Roscoe and his co-workers at Cambridge (England), suggests that plastic volume changes may be handled independently from those for shear distortion, using normality rule. It was implied that the materials considered conforms with the concept of stability introduced by Drucker (1959).

Further development was made by Roscoe and Schofield (1963), who maintained that during unloading the change in void-ratio is elastic and dependent solely on the change in average principal effective stress $p' = \frac{1}{3}(\sigma_1' + 2\sigma_3')$ and is affected by change in deviator stress $q = \sigma_1 - \sigma_3$.

Roscoe and Poorooshasb (1963) believed that most of the disagreement was caused by the inability of the testing methods used to reveal the true behaviour of soils under stress. By careful experimentation in restricted fields it is possible to obtain reliable data which removes some of the previous apparent contradictions and to obtain improved agree ment with theory.

They showed that for normally consolidated samples of remoulded saturated clays, the strains observed during triaxial compression tests on samples with different degrees of drainage may be related to each other and the imposed stresses provided certain restrictive conditions are satisfied.

Stress was treated as an independent variable and the change in arbitrarily selected strain parameters expressed as functions of this variable.

2.2 CAM CLAY MODEL

Due to the continuous effort by Roscoe and his co-workers (Cambridge group), a model which is known as Cam-clay model was presented. The concept of unique state boundary surface is introduced. The state of sample is defined by $p' = \frac{1}{3}(\sigma'_1+2\sigma'_3)$ $q = (\sigma_1-\sigma_3)$ and a third variable e (void ratio). The state of sample can be represented by a point in p, q, e space. It has been found that when stress paths for triaxial compression tests on normally consolidated samples of saturated remoulded clay are plotted in p', q, e space, they form a unique surface called state boundary surface. State boundary surface separates those states which are accessible to a given clay from those which are not.

Roscoe and his co-workers made two assumptions concerning the elastic or recoverable behaviour of wet clays during shear.

(1) First they assumed that, the elastic surface or wall (which cuts the q = 0 plane in p', q, e. space along the swelling curve) is vertical so that the elastic limit curve lies directly above the swelling curve (This is the Calladine's assumption).

(2) Secondly they assumed that there is hardly any recoverable energy associated with shear distortion

$$\text{i.e.} \quad \delta\varepsilon_r = 0 \qquad (1.2)$$

and at all times

$$\delta\varepsilon = \delta\varepsilon^p \qquad (2.2)$$

The volumetric strain is only a function of $p' = \frac{1}{3}(\sigma'_1+\sigma'_2+\sigma'_3)$

$$\delta v^e = \frac{\kappa}{1+e} \frac{dp}{p'} \qquad (3.2)$$

where κ is the slope of the swelling curve in e-log p plot.

Based on the above assumptions Cam-clay model was derived and the following relations were obtained. Roscoe et al (1958) Incremental volumetric strain δv is given by

$$\delta v = \frac{1}{1+e}\left[\frac{\lambda-\kappa}{M}\delta\eta + \lambda\frac{\delta p}{p'}\right] \qquad (4.2)$$

Incremental shear strain $\delta\varepsilon$ is given by

$$\delta\varepsilon = \frac{\lambda-\kappa}{1+e}\left[\frac{p'\delta\eta + M\,\delta p}{Mp'(M-\eta)}\right] \qquad (5.2)$$

yield locus is given by

$$\eta = M \log\frac{p_o}{p_e} \qquad (6.2)$$

State Boundary surface is given by

$$\eta = \frac{M}{1-\frac{\kappa}{\lambda}} \log\frac{\bar{p}'}{p_e} \qquad (7.2)$$

where $\bar{p}'$ is the value p when $\eta = 0$ and p_e is the equivalent consolidation pressure as proposed by Hvorslev (1937).

2.3 MODIFICATION TO CAM. CLAY MODEL

Burland (1965) used a different function for dissipated work δw as

$$\delta w = p' \sqrt{(\delta v^p)^2 + (M\,\delta\varepsilon^p)^2} \tag{8.2}$$

and modified the cam. clay model. In this modified form, the expression for volumetric strain δv is given by

$$\delta v = \frac{1}{1+e}\left[(\lambda-\kappa)\frac{2\eta\,d\eta}{M^2+\eta^2} + \lambda\frac{dp}{p'}\right] \tag{9.2}$$

and the expression for the shear strain $\delta\varepsilon$ is given by

$$\delta\varepsilon = \frac{\lambda-\kappa}{1+e}\left[\frac{2\eta}{M^2+\eta^2}\right]\frac{2\eta\,d\eta}{M^2+\eta^2} + \frac{\delta p}{p'} \tag{10.2}$$

The yield locus is given by

$$\frac{p'}{p_o} = \frac{+\,M^2}{M^2+\eta^2} \tag{11.2}$$

State boundary surface is given by

$$\frac{p}{p_e} = \left[\frac{M^2}{M^2+\eta^2}\right]^{(1-\lambda/\kappa)} \tag{12.2}$$

2.4 WROTH'S MODEL

Wroth (1968) suggested that it is possible to predict the stress strain behaviour of n.c. clays from the results of two series of tests

(1) η-constant tests (anisotropic consolidation tests)

(2) p'-constant (pure shear test)

The following expressions were derived out for this model.
Incremental volumetric strain δv is given by

$$\delta v = \frac{\partial v}{\partial p}\,dp + \frac{\partial v}{\partial \eta}\,d\eta \qquad (13.2)$$

Incremental shear strain ϵ is given by

$$\delta\varepsilon = \frac{\partial \varepsilon}{\partial p}\,dp + \frac{\partial \varepsilon}{\partial \eta}\,d\eta \qquad (14.2)$$

The factors $\frac{\partial v}{\partial p}$ and $\frac{\partial \varepsilon}{\partial p}$ were obtained from. η Constant tests and the factors $\frac{\partial v}{\partial \eta}$ and $\frac{\partial \varepsilon}{\partial \eta}$ were obtained from p-constant tests.

This model has the ability to predict both loading and stress-paths, provided the parameters are obtained in the corresponding manner. Wroth has predicted the results of a a few stress path tests conducted on isotropically consolidated samples but not on κ_0 consolidated samples.

2.5 NEWLAND (1973)

Newland argued that any stress path may be considered to be a combination of the two phases. Undrained shear will q-increasing (p-constant) and consolidation with q-constant. The results obtained by Newland on anisotropically consolidated kaolin could not be predicted by the Cambridge model. He brought the dependence of $\frac{\kappa}{\lambda}$ on consolidation stress and the effect of initial η value of λ. From his experimental results for special stress paths investigated, he developed a semi-empirical relation between $\frac{\varepsilon}{\varepsilon_v}$ and η.

2.6 MATHUR'S PROPOSAL (1976)

Mathur (1976) developed a model based on p-constant and q-constant tests and arrived at conclusions similar to those proposed by Newland. The basic equations in this model are follows.

Incremental volumetric strain δv is given by

$$\delta v = \frac{\partial v}{\partial p}\, dp + \frac{\partial v}{\partial q}\, dq \tag{15.2}$$

Incremental shear strain $\delta\varepsilon$ is given by

$$\delta\varepsilon = \frac{\partial\varepsilon}{\partial p}\, dp + \frac{\partial\varepsilon}{\partial q}\, dq \tag{16.2}$$

The parameters $\frac{\partial v}{\partial p}$ and $\frac{\partial\varepsilon}{\partial p}$ are obtained from q-constant tests and the parameters $\frac{\partial v}{\partial q}$ and $\frac{\partial\varepsilon}{\partial q}$ are obtained from p-constant tests.

This model showed good agreement between predictions and observations on anisotropically consolidated samples, tested under varieties of stress paths.

CHAPTER 3

TEST PROGRAMME

3.1 GENERAL

Fully drained triaxial tests were carried out on specimens (11.68 cm^2 area) consolidated under 0.2 kg/cm^2 vertical pressure in perspex glass cylinders. The tests were designed to provide information on yield curve and the failure states of the clay in the stress region applicable to most Civil Engineering problems. The results are presented interms of the following effective stress and strain parameters.

(1) Mean normal stress $p' = \dfrac{(\sigma_1' + 2\sigma_3')}{3}$

(2) Deviatoric stress $q = (\sigma_1' - \sigma_3')$

(3) Volumetric strain increment $\delta v = \delta\varepsilon_1 + 2\delta\varepsilon_3$

(4) Shear $\delta\varepsilon = \frac{1}{2}(\delta\varepsilon_1 - \delta\varepsilon_3)$

Axial displacements and volumetric changes were measured during all phases of testing. Strain parameters are calculated from these measurements by assuming a right circular cylindrical specimen shape. The total strains are calculated by simple summation of the natural strain increments.

All tests were carried out at constant temperature and under a back pressure of $2kg/cm^2$. Filter paper strips were used to facilitate drainage. Load increment durations were well in excess of the time required for complete dissipation of excess pore water pressures.

p constant tests and q constant tests were performed on anisotropically consolidated specimens. In addition to that conventional triaxial tests (both drained and undrained) were performed on anisotropically and isotropically consolidated samples.

3.2 MATERIAL USED

The soil used in this study was a mixture of commercially available Kaolinite and Montmorilonite at a proportion of 80 % to 20 %. Grain size analysis was done for this mixture and it was found the soil contains 80% clay. The liquid limit is 83.5% and the plastic limit is 33.5% giving a plasticity index of 50. The specific gravity of this mixture is 2.6.

3.3 PREPARATION OF SPECIMEN

The soil was mixed with drained distilled water to form a slurry approximately having a water content twice that of LL, and this slurry was reserved in a glass container, so that all the specimens could be prepared from the same slurry. Specimens of 11.68 cm^2 area, were sedimented

in perspex glass cylinder from that slurry. Specimens were consolidated to a predetermined pressure (0.2 kg/cm^2).

3.4 TRIAXIAL EQUIPMENT

Standard triaxial cells of the Norwegion type were used. The pressure lines were connected to a self compensating mercury column system, so that pressures applied during the test could be maintained constant. The tests were performed in the triaxial testing room, which was maintained at constant temperature around 25^{o}c to eliminate the effect of temperature variation.

In the undrained tests perspex null indicators were used for the measurement of pore pressures. Aluminium frame hangers were used to support dead load in all of the anisotropic consolidation and shear tests.

3.5 SET UP OF SPECIMEN

Before setting up the specimen all the pressure lines connected to the base of the cell were flushed with deaired water to eliminate entrapped airbubbles. The specimen was then put on a porous stone which was inturn on the pedastal of the cell. Another porous stone and a loading cap were put on top of it. Both porous stones had been boiled and saturated before hand.

Eight 1 x 8 cm Watman's No. 42 filter paper strips were placed around the specimens at approximately equal spacing. The use of filter side drains was to increase the rate of consolidation and equalization of pore pressures. The ends of the strips were placed in contact with the porous stone. Two thin rubber membranes with a thin layer of silicon grease applied in between were used to enclose the specimen. Constant head mercury pot arrangement was used to apply constant lateral stress.

3.6 SATURATION OF SPECIMENS

To ensure complete saturation and to overcome difficulties in measuring negative pore pressures a back pressure of $2kg/cm^2$ was applied to the specimen. In the saturation process back pressure and cell pressures were applied equally and simultaneously with a pressure screw control at a slow rate to minimize the effect of disturbance during pressure application.

The amount of water imbibed by the specimen during the saturation process and testing was measured by a volume measuring unit having a least count approximately 0.01 cc.

3.7 ANISOTROPIC CONSOLIDATION

Anisotropic consolidation was arrived out by applying a vertical stress to the sample during consolidation in

addition to the cell pressure. Dead loading was used for this purpose. It was decided to apply pressures in small increments to avoid significant porepressure gradient. The time required for the consolidation process varied from 3 to 8 days.

All specimens were first consolidated anisotropically under various η condition to a certain predetermined pressure. The following equation was used to calculate the dead load increments for a corresponding cell pressure σ_3, to produce anisotropic consolidation. The same equation was used to calculate the dead load increment in shear process by using the η values at every stage.

$$(\sigma_1' - \sigma_3')\ a = W + W_h + W_r - a_r \cdot \sigma_3 \qquad (1.3)$$

substituting

$$\frac{\sigma_1' - \sigma_3'}{(\sigma_1' + 2\sigma_3')/3} = \eta \qquad (2.3)$$

we will get

$$W = \frac{3\eta}{(3-\eta)} \cdot a \cdot (\sigma_3 - u) - (W_h + W_r - a_r\sigma_3) \qquad (3.3)$$

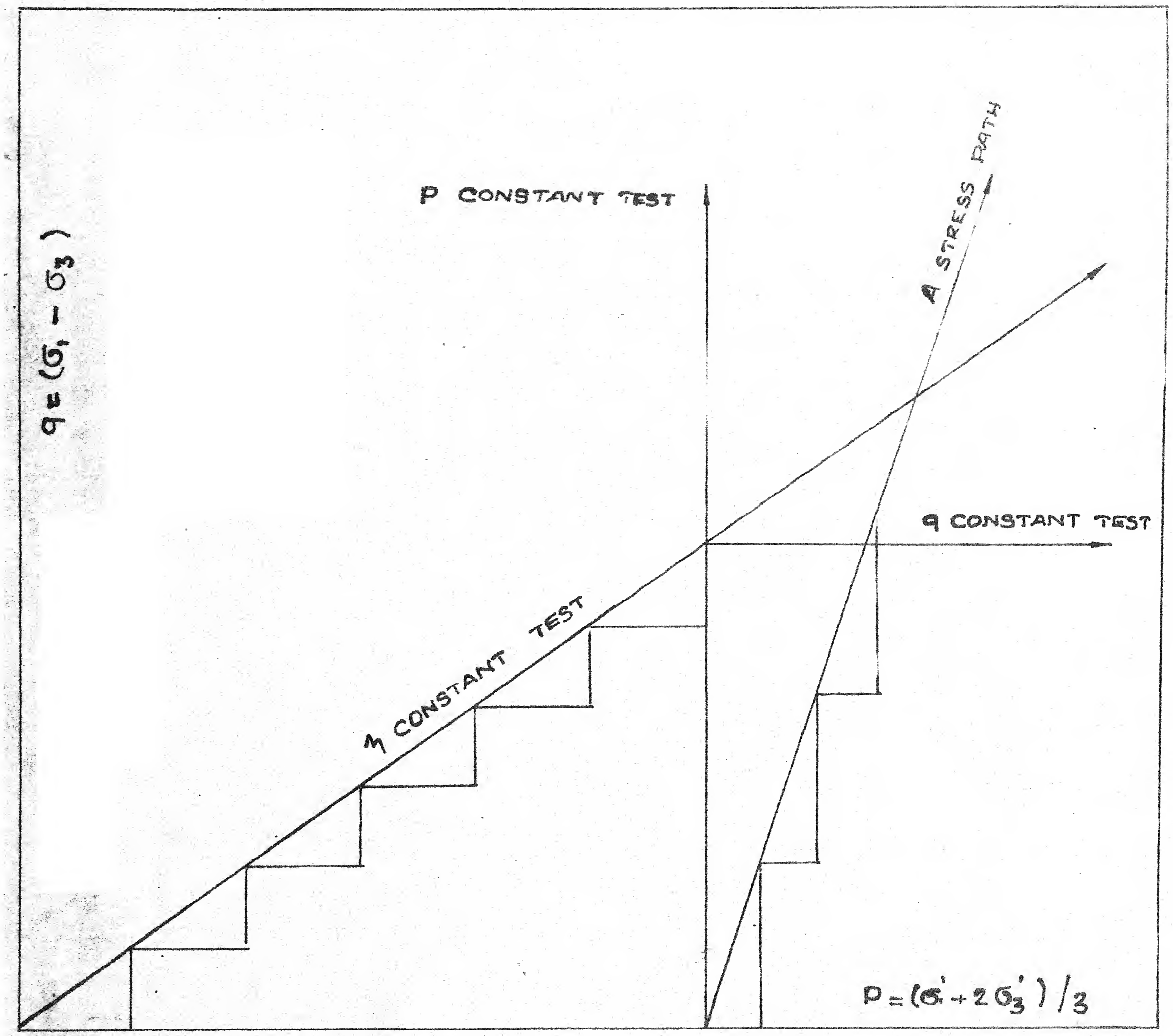
where a .... average area of the specimen

a_r.... area of the piston

W_h.... weight of the hanger

W_r.... weight of the piston

W .... dead load to be added.



q = (σ1 − σ3)
P CONSTANT TEST
A STRESS PATH
q CONSTANT TEST
η CONSTANT TEST
P = (σ1' + 2σ3')/3

FIG 1

3.8 SHEAR PROCESS

3.8.1 p-constant tests

Samples were first consolidated anisotropically to 2.5 kg/cm^2 of mean effective pressure along various lines. q was increased in small steps, while keeping the p' at the constant value 2.5 kg/cm^2.

$$p' = \frac{1}{3}(\sigma_1 + 2\sigma_3) - u \qquad (4.3)$$

where u is the pore pressure.

$$q = (\sigma_1 - \sigma_3) \qquad (5.3)$$

Using (4.3) and (5.3) the value of σ_3 for a particular p', q combination was calculated. By simply dividing q by p' the corresponding η value was obtained. Using (3.3) the dead load to be used was calculated.

3.8.2 q constant tests

Samples were first consolidated anisotropically to 2.5 kg/cm^2 of mean effective pressure, along various η lines. p' was increased in small steps while keeping q at the constant value, which was reached at the end of the anisotropic process. Dead load at each step was calculated as explained under 3.8.1.

3.8.3. CONVENTIONAL DRAINED TESTS.

Samples were first consolidated anisotropically to 2.5 kg/cm^2 of mean effective pressure along various η lines.

q was increased in small steps while keeping σ_3 at the constant value, reached at the end of the anisotropic consolidation process.

$$\Delta q = 3 \Delta p \tag{6.3}$$

Using (6.3), (5.3) and (4.3) the value of σ_3 for a particular p', q combination was calculated. Using (3.3) the dead load was calculated.

3.8.4 Special shear tests

One specimen, initially consolidated anisotropically was tested along a stress path which consists of anisotropic consolidation, q-constant consolidation, p-constant shear and conventional drained shear test.

CHAPTER 4

OBSERVED STRESS DEFORMATION BEHAVIOUR OF ANISOTROPICALLY CONSOLIDATED CLAY

4.1 GENERAL

Most of the data on the behaviour of saturated clay has been obtained from the results of tests on specimens consolidated under an anisotropic stress system at various n values. In the ground, however, sedimentary clay strata have been consolidated under conditions of no lateral strains and in order to understand the field behaviour of natural deposits, it is necessary to study laboratory specimens which have been consolidated under these conditions.

The ratio of the horizontal to the vertical effective stress, which satisfies the condition of zero lateral strain is called the coefficient of earth pressure, at rest, and the symbol K_o is used for this ratio. The determination of K_o from triaxial test is presented.

4.2 RELATIONSHIP BETWEEN VOID RATIO AND MEAN EFFECTIVE PRESSURE

Remoulded specimens of clay were consolidated in the triaxial apparatus under anisotropic and isotropic conditions. The test results have been plotted in e vs ln p' plot as shown in Fig. 2. The failure points from drained undrained shear tests are also shown in Fig. 2.

Table gives the values of λ (compression index) obtained from various anisotropic consolidation tests along with the value of κ (swelling index) obtained from isotropic swelling test.

Table 1

η	λ	κ
0	.291	0.065
.2	.287	-
.4	.280	-
.6	.264	-
.8	.252	-

a $\frac{\kappa}{\lambda}$ value of 0.224 was obtained for isotropic case. It may be seen (table 1) that the slope of the consolidation line consistently decreases as the anisotropic ratio η increases. The difference in λ values between $\eta = 0$ and $\eta = .8$ cases is 13.5% . This is comparable with the 15% variation in λ values between $\eta = 0$ and $\eta = .9$ as obtained by Newland (1973).

The dependence of λ on the value of η in the drained tests introduces an added complication to the simple concept of a unique state boundary surface and in

the prediction of stress deformation behaviour of clays. This aspect would be considered later.

4.3 UNDRAINED SHEAR TESTS

It is of some interest to examine the difference in the pore water pressure characteristics of the specimens consolidated under different conditions. These characteristics are, of course, implicit in the stress paths, but it is nevertheless useful to draw attention to the values of the 'A' parameters found in the tests.

After isotropic consolidation the average value of parameter 'A' in compression tests, defined as

$$\frac{\Delta u_f - \Delta\sigma_{3f}}{(\Delta\sigma_1 - \Delta\sigma_3)_f}$$ was found to be close to unity. [Table 2]

and in the range commonly associated with normally consolidated clays. For the K_o consolidated specimens, however the average value of A_f was found to be 1.6. This is very close to the value $A_f = 1.8$ for the K_o consolidated specimens on Weald clay, obtained by Henkel (1963).

Fig. 6 and 7 illustrate the stress strain behaviour and the pore pressure build up in the undrained tests performed. Table 2 shows the pore pressure parameters A_f obtained from the above tests.

TABLE 2

Nature of the sample	Initial consolidation pressure	A_f	Voidratio at failure
ISOTROPIC	1 kg/cm^2	.98	1.31
ISOTROPIC	2 ''	.99	1.11
ISOTROPIC	3 ''	.98	0.98
ANISOTROPIC	1 ''	1.58	1.27
ANISOTROPIC	2 ''	1.53	1.08
ANISOTROPIC	3 ''	1.65	0.97

Results of undrained tests on both isotropically and anisotropically consolidated samples will be used while discussing the state boundary surface for this clay.

4.4 RESULTS OF P'-CONSTANT TESTS

P-constant tests were performed on the specimens which were anisotropically consolidated along various lines initially. The results are plotted in Figs. 3 and 4. Fig. 4 shows that all specimens consolidated to 2.5 kg/cm^2 mean effective pressure are consistently failing at a q value between 2.4 kg/cm^2 and 2.5 kg/cm^2. This gives the average M = 0.98 and the angle of internal friction $\bar{\phi} = 24.5^{\circ}$.

4.5 q-CONSTANT TESTS

q-constant tests were performed on anisotropically consolidated specimens. The results are presented in Fig. 5. Two things are clearly seen from this results.

(1) Each curve of q-constant phase is flatter at the start and then become steep so that an equivalent pre-consolidation pressure P_c can be obtained. Table 3 gives the values of P_c obtained in different cases

TABLE 3

INITIAL P'	CONDITIONS q	η	P_c kg/cm^2
2.5	1.0	.4	2.75
2.5	1.5	.6	2.95
2.5	2.0	.8	3.15

(2) Due to the previous stress history some of the curves have a tendency to cross the virgin consolidation curve at high pressures.

4.6 CONVENTIONAL DRAINED TESTS

Conventional drained tests were performed on the isotropically and anisotropically consolidated samples and the results are shown in Figs. 19, 20 and 21. These results were used to study the predictability at the proposed model.

4.7 SPECIAL TEST

One specimen was tested along a stress path which incorporates

1. Anisotropic consolidation
2. q-constant consolidation
3. p′-constant shear test
4. conventional drained shear test

in series and the result was used to check the predictability of the proposed model.

4.8 VOLUMETRIC YIELD POINT DETERMINATION

Fig. 12 and 13 show the results obtained in drained tests plotted in the form of q vs. volumetric strain in order to get the volumetric yield points. Table 4 gives the details of these points. These yield points are shown in Fig. 10 with the stress paths.

TABLE 4

INITIAL CONDITION			YIELD CONDITIONS			Remarks
η	p'	q	p'	q	ε_v%	
.2	2.5	.5	2.5	1.20	.57	P-constant tests
.4	"	1.0	"	1.77	.51	
.6	"	1.5	"	2.04	.36	
.8	"	2.0	"	2.19	.20	
.4	"	1.0	2.75	1.76	2.40	Conventional drained test

Tests on isotropically consolidated samples also indicate a yield point but the fundamental basis for this behaviour is not quite clear.

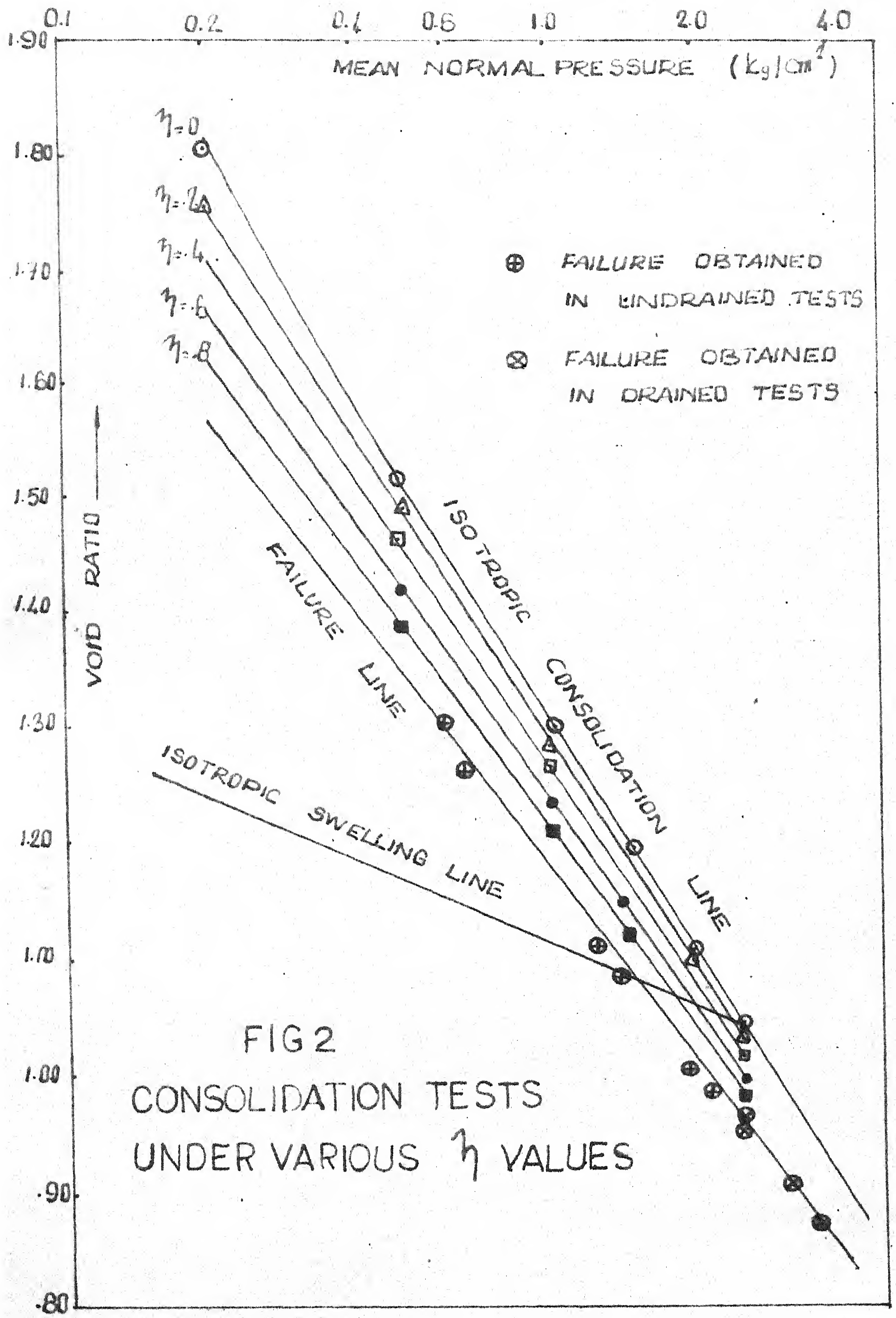
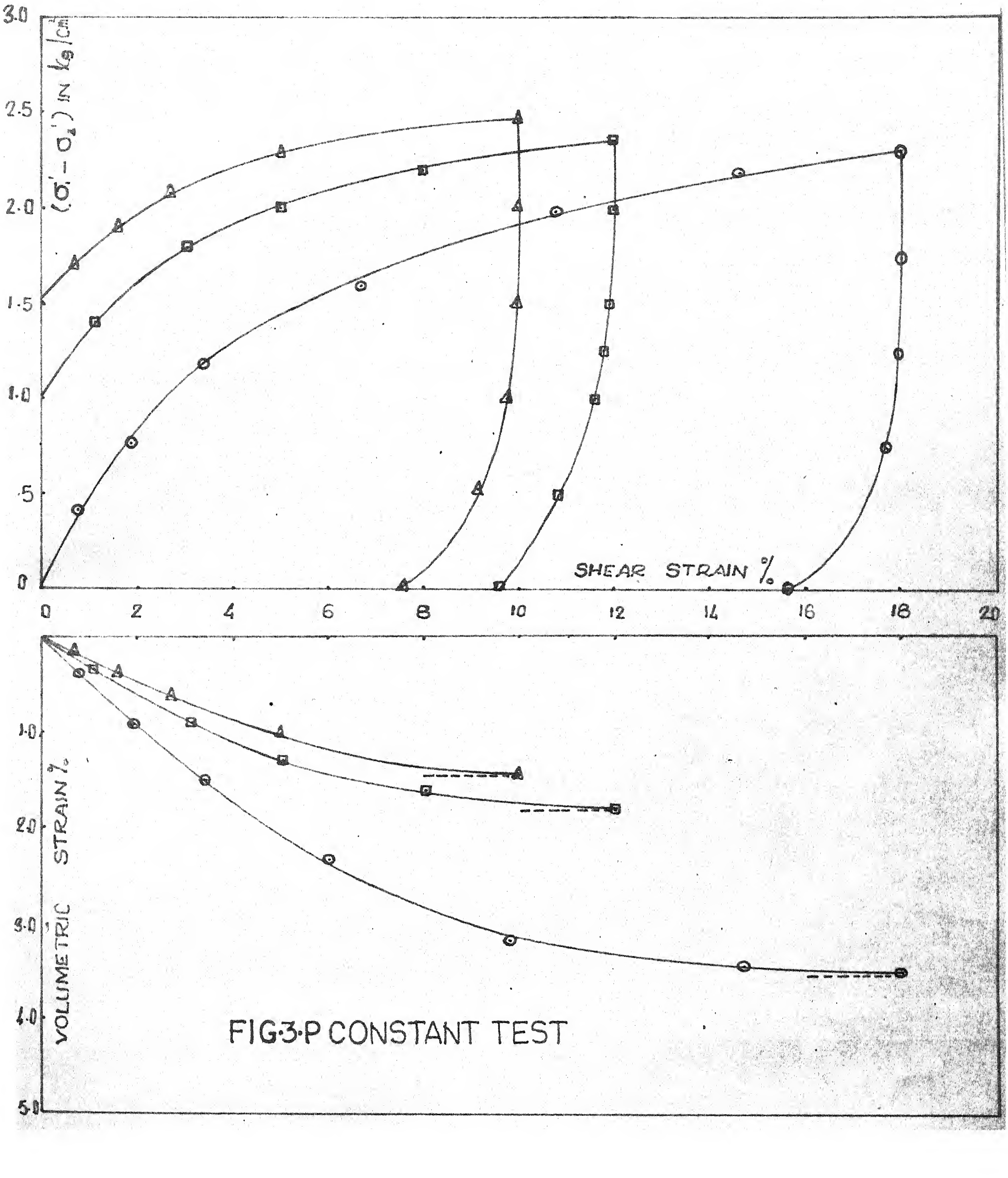


FIG 2

CONSOLIDATION TESTS UNDER VARIOUS η VALUES



FIG·3·P CONSTANT TEST

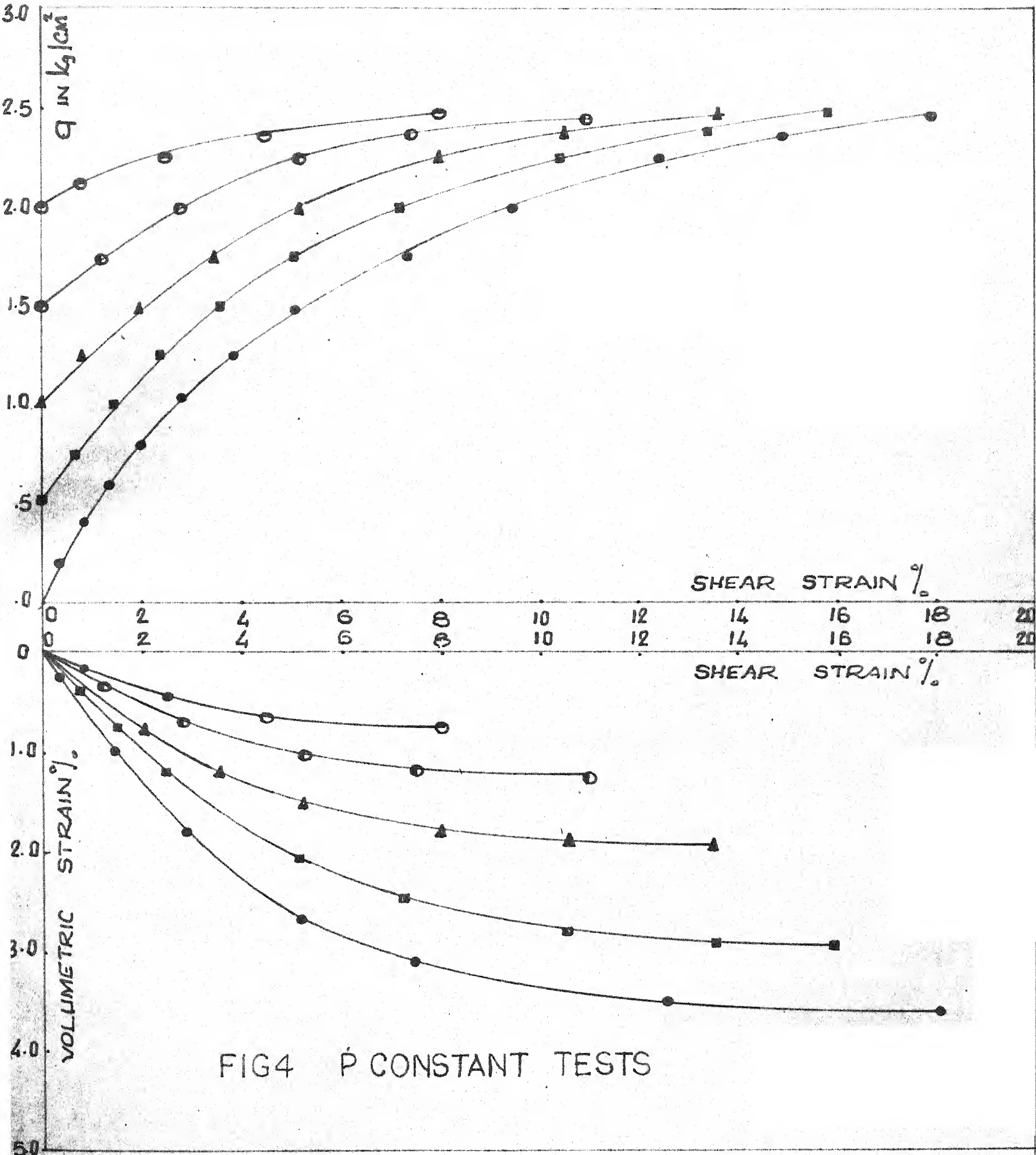


FIG4 P CONSTANT TESTS

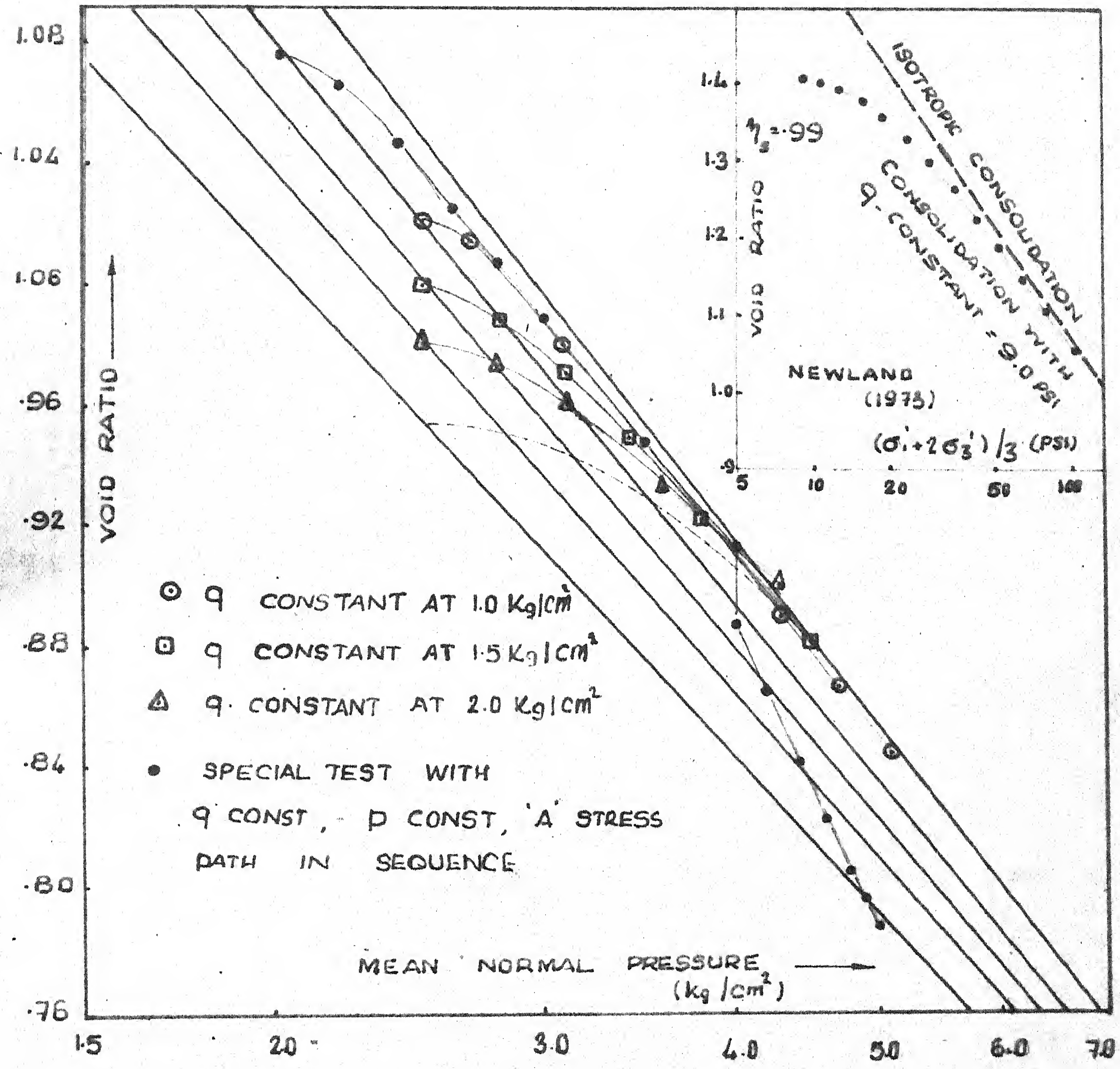


FIG 5 q-CONSTANT TESTS

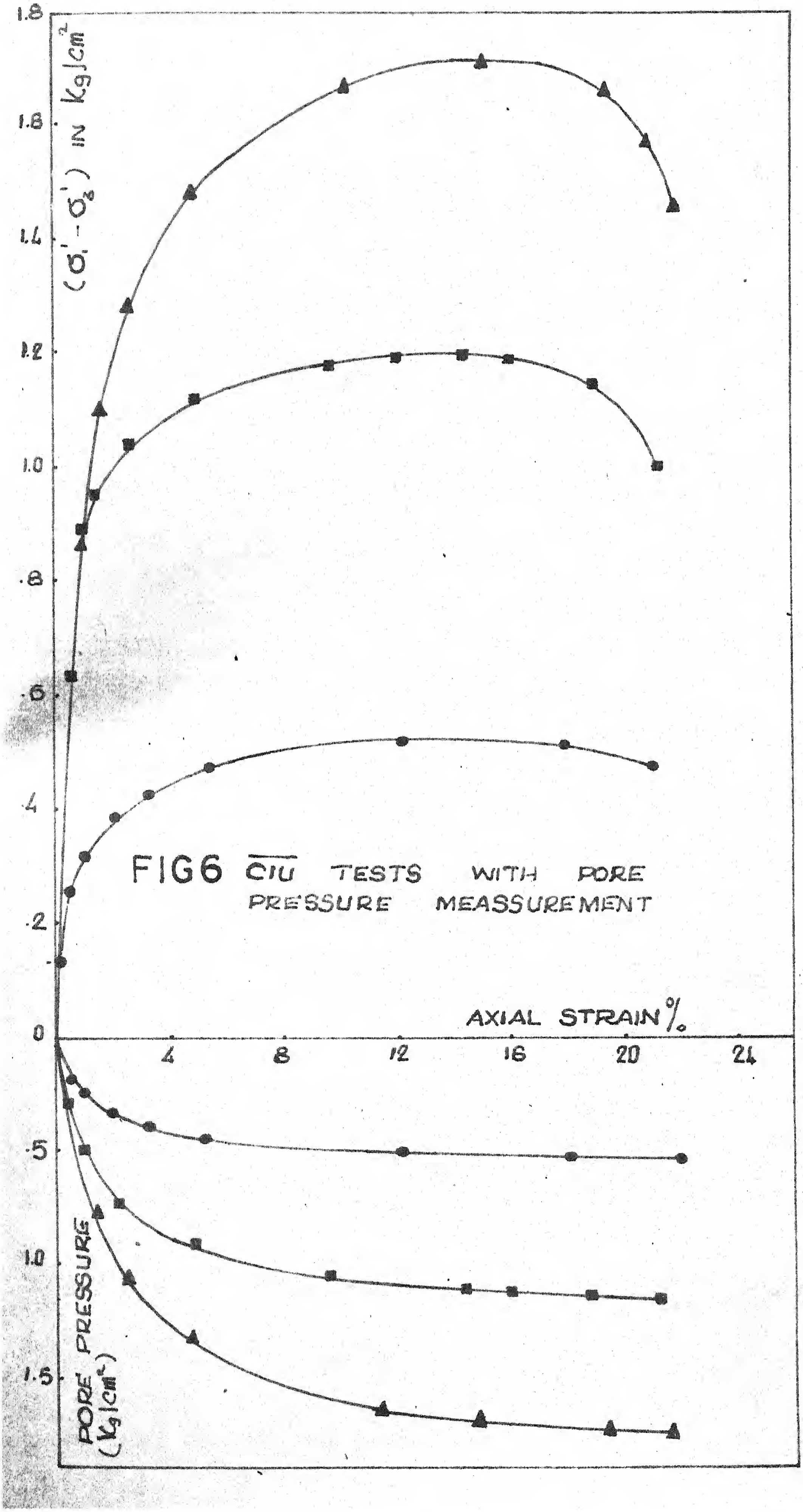


FIG6 $\overline{CIU}$ TESTS WITH PORE PRESSURE MEASSUREMENT

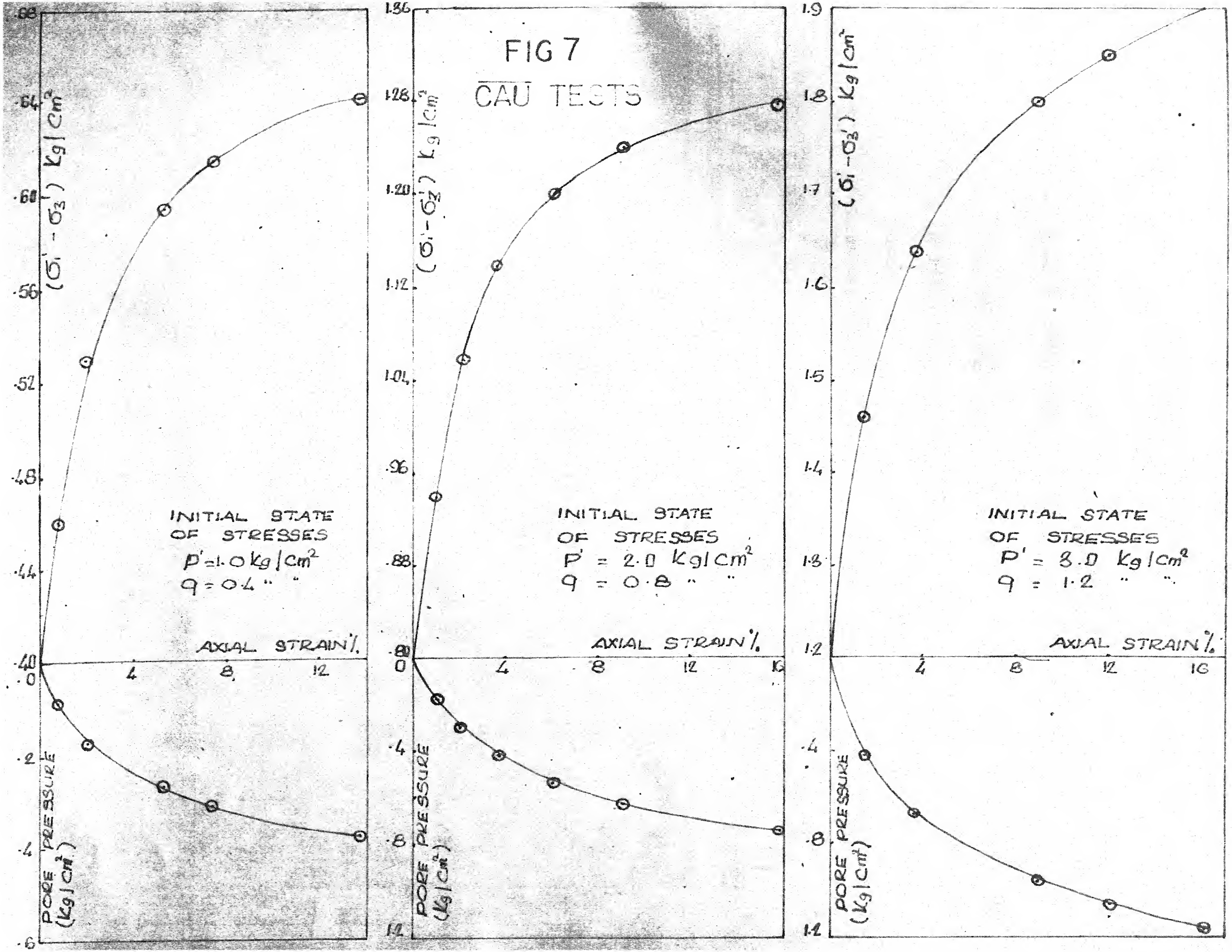


FIG 7
CAU TESTS
$(\sigma_1' - \sigma_3')$ Kg/cm²
INITIAL STATE
OF STRESSES
P' = 1.0 Kg/cm²
q = 0.4 " "
AXIAL STRAIN %
PORE PRESSURE (Kg/cm²)
INITIAL STATE
OF STRESSES
P' = 2.0 Kg/cm²
q = 0.8 " "
INITIAL STATE
OF STRESSES
P' = 3.0 Kg/cm²
q = 1.2 " "

CHAPTER 5

PREDICTED AND OBSERVED STRESS STRAIN BEHAVIOUR

5.1 PROPOSED MODEL BY MATHUR (1976)

In this model Mathur proposed to analyse the drained shear behaviour along any stress path by considering the contributions made by p'-constant and q=constant tests.

Since p' & q are the independent variables in this approach strains (both volumetric and shear) are considered to be the functions of p' and q only

$$\text{volumetric strain} \quad \varepsilon_v = f\,(p', q) \qquad (1.5)$$

$$\text{and} \quad \text{shear strain} \quad \varepsilon = g\,(p', q) \qquad (2.5)$$

Now during the application of any stress probe (dp, dq), the changes in the strains can be represented by the following relationships

$$d\varepsilon_v = \frac{\partial \varepsilon_v}{\partial p}\,dp + \frac{\partial \varepsilon_v}{\partial q}\,dq \qquad (3.5)$$

$$d\varepsilon = \frac{\partial \varepsilon}{\partial p}\,dp + \frac{\partial \varepsilon}{\partial q}\,dq \qquad (4.5)$$

$\frac{\partial \varepsilon_v}{\partial p}$ and $\frac{\partial \varepsilon}{\partial p}$ are the parameters, respectively giving the part of volumetric and shear strains, contributed by q-constant consolidation process, and these parameters are obtained from q-constant consolidation tests

$\frac{\partial \varepsilon_v}{\partial q}$ and $\frac{\partial \varepsilon}{\partial q}$ are the parameters respectively giving the

part of strains contributed by shear process and these parameters are obtained from p-constant pure shear tests.

5.2 DETERMINATION OF PARAMETERS

p'-constant and q-constant tests were performed on anisotropically consolidated specimens. The p-constant test results were plotted in

$$\ln \frac{q_f}{q_f - q} \quad \text{vs} \quad \text{shear strain} \qquad \text{(fig. 8)}$$

$$\text{and} \quad \frac{q}{q_f} \quad \text{vs volumetric strain} \qquad \text{(fig. 11)}$$

plots as suggested by Wroth (1968). The slope of these straight lines known as D' is related to the parameters $\frac{\partial \varepsilon_v}{\partial q}$ and $\frac{\partial \varepsilon}{\partial q}$ in the following way

$$\frac{\partial \varepsilon_v}{\partial q} = \frac{D'}{q_f} \qquad (5.5)$$

$$\frac{\partial \varepsilon}{\partial q} = \frac{D'}{q_f - q} \qquad (6.5)$$

Table 5 gives the values of D' obtained from figs. 8 for the samples having different initial η values.

Calculated mean for the above values of D' = .046, having a standard deviation of $\pm$ 16 % .

Table 5(fig. 8)

Initial n	D'
0	.055
.2	.047
.4	.047
.6	.041
.8	.039

Table 6 (fig. 11)

Initial n	D'
0	.0465
.2	.0460
.4	.0455
.6	.0460
.8	.0455

Table 6 gives the values of D' obtained from fig. 11 for the sample having different initial n values. Calculated mean for the above values of D is .046, having a standard deviation $\pm$ 2% .

The results of q-constant tests were plotted as

$$\left.\begin{array}{l} \text{Volumetric strain } (\epsilon_v) \text{ vs. } \ln p' \\ \text{and shear strain } (\epsilon) \text{ vs. } \ln p' \end{array}\right\} \text{ in Fig. 9.}$$

The slope of the plot ϵ_v vs $\ln p'$ is α_1 and is related to the parameter $\frac{\partial \epsilon_v}{\partial p}$ in the following way (7.5)

$$\frac{\partial \epsilon_v}{\partial p} = \frac{\alpha_1}{p'} \qquad (7.5)$$

The slope of the plot ϵ vs. $\ln p'$ is α_3 and is related to the para meter $\frac{\partial \epsilon}{\partial p}$ in the following way

$$\frac{\partial \epsilon}{\partial p} = \frac{\alpha_3}{p'} \qquad (8.5)$$

From fig. 9 α_1 = .107 and α_3 = .035 .

From the equation (3.5), (4.5), (7.5) and (8.5) the incremental volumetric and shear strains can be written as

$$d\epsilon_v = \alpha_1 \frac{dp}{p'} + \frac{D'}{q_f}\,dq \qquad (9.5)$$

$$d\epsilon = \alpha_3 \frac{dp}{p'} + \frac{D'}{q_f - q}\,dq \qquad (10.5)$$

for stress paths in which both Δp and Δq are increasing.

5.3 PREDICTED AND OBSERVED STRESS PATH

Using $\alpha_1 = .107$ $\alpha_3 = .035$ and $D' = .046$ in (9.5) and (10.5), we get

$$d\epsilon_v = 0.107 \frac{dp}{p'} + .046 \frac{dq}{q_f} \qquad (11.5)$$

$$d\epsilon = .035 \frac{dp}{p'} + .046 \frac{dq}{q_f - q} \qquad (12.5)$$

Table 7 and 8 show the specimen calculation for the predicted stress strain behaviour, along 'A' stress path (conventional drained test).

TABLE 7

p'	q	q_f	$.107 \frac{dp}{p'}$	$.046 \frac{dq}{q}$	$d\epsilon_v$ %	ϵ_v %
2.5	0.00	2.45	–	–	–	–
2.6	0.30	2.55	42×10^{-4}	55.1×10^{-4}	.971	.971
2.7	0.60	2.65	40.5×10^{-4}	53.1×10^{-4}	.936	1.907
2.8	0.90	2.75	39.0×10^{-4}	51.1×10^{-4}	.901	2.808

TABLE 8

p'	q	q_1-q	$.035\,\frac{dp}{p'}$	$.046\,\frac{dq}{q_1-q}$	$d\epsilon$	ϵ
2.5	0.00	2.45	-	-	-	-
2.6	0.30	2.25	13.7×10^{-4}	58.8×10^{-4}	.725	.73
2.7	0.60	2.05	13.2×10^{-4}	64.3×10^{-4}	.775	1.50
2.8	0.90	1.85	12.7×10^{-4}	70.8×10^{-4}	.835	2.34

At lower stress levels the predicted value of axial strain slightly higher than that of observed value. But at higher stress level the predicted value of axial strain is slightly lower than that of observed value. However, the maximum difference observed between the predicted and observed value is less than 2 % . This maximum difference is still less in the case of anisotropically consolidated samples. The maximum difference between the predicted and observed strains for anisotropically consolidated sample is less than 1% .

As far as the volumetric strain is concerned the proposed model predicts slightly lower value than the experimental . But the maximum difference is not more than 1% .

5.4 PREDICTION OF ANISOTROPIC CONSOLIDATION BEHAVIOUR

By definition anisotropic consolidation test is a test in which the ratio between major and minor principal stress is always same.

$$\text{i.e.} \quad \frac{\sigma_3'}{\sigma_1'} = K$$

$$\sigma_1' - \sigma_3' = (1-K)\ \sigma_1' \tag{13.5}$$

$$\frac{1}{3}(\sigma_1' + 2\sigma_3') = \frac{1}{3}(1+ 2K)\ \sigma_1'$$

$$\frac{q}{p'} = \frac{\sigma_1' - \sigma_3'}{\frac{1}{3}(\sigma_1' + 2\sigma_3')} = \frac{3(1-K)}{(1- 2K)} = \eta \tag{14.5}$$

$$dq = \eta\ dp' \tag{15.5}$$

Now $q_f = M.p'$ (16.5)

Using (14.5), (15.5), (16.5) in (3.5) and (4.5) we get

$$d\epsilon_v = \alpha_1 \frac{dp}{p'} + \frac{D'\eta}{MP'}\ dp \tag{17.5}$$

$$d\epsilon = \alpha_3 \frac{dp}{p'} + \frac{D'\eta}{M-\eta}\ dp \tag{18.5}$$

or by putting $d\epsilon_v = \dot{\epsilon}_v$ and $d\epsilon = \dot{\epsilon}$

$$\dot{\epsilon}_v = [\ \alpha_1 + \frac{D'\eta}{M}\ ]\ \frac{dp}{p'} \tag{19.5}$$

$$\dot{\epsilon} = [\ \alpha_3 + \frac{D'\eta}{M-\eta}\ ]\ \frac{dp}{p'} \tag{20.5}$$

The flow rule for anisotropic consolidation becomes

$$\frac{\dot{\epsilon}}{\dot{\epsilon}_v} = \frac{\alpha_3 + \frac{D'\eta}{M-\eta}}{\alpha_1 + \frac{D'\eta}{M}} \tag{21.5}$$

Substituting $\alpha_1 = .107$, $\alpha_3 = .035$ and $D' = .046$ we get

$$\frac{\dot{\epsilon}}{\dot{\epsilon}_v} = \frac{.035 + \frac{.046\,\eta}{(.98 - \eta)}}{.107 + \frac{.046\,\eta}{.98}} \tag{22.5}$$

Equation (22.5) predicts the total strain increment ratio $\frac{\dot{\epsilon}}{\dot{\epsilon}_v}$ for anisotropic consolidation tests for various η values. Fig. 16 shows the prediction of the flow rule by the proposed method along with the predictions by two other models

TABLE 9

η	observed $\frac{\dot{\epsilon}}{\dot{\epsilon}_v}$	predicted $\frac{\dot{\epsilon}}{\dot{\epsilon}_v}$
0	-	.32
.2	.24	.42
.4	.58	.54
.6	1.08	.90
.8	2.40	1.62

Table 9 gives the observed $\frac{\dot{\epsilon}}{\dot{\epsilon}_v}$ along various consolidation lines, with the predicted value by the equation (22.5). The predictions seems to be very close to the observed values. Roscoe and Burland (1968) over predict the value of $\dot{\epsilon}/\dot{\epsilon}_v$ as shown in fig. (16). In case of soils the direction of plastic strain increment vector is dependent on the direction of stress increment vector, and the flow rule used by Roscoe and Burland does not incorporate this important fact.

5.5 PREDICTION OF K_o BY THE PROPOSED MODEL

By substituting $\frac{\epsilon}{\epsilon_v} = \frac{2}{3}$ for no lateral strain condition in (21.5) we get

$$\eta^2 + \frac{M\alpha_1}{D'}\left[1 + .5\frac{D'}{\alpha_1} - 1.5\frac{\alpha_3}{\alpha_1}\right]\eta +$$

$$\left[1.5\frac{\alpha_3}{\alpha_1} - 1\right] M^2 \cdot \frac{\alpha_1}{D'} = 0 \qquad (23.5)$$

Using $D' = .046$, $\alpha_1 = .107$ and $\alpha_3 = .035$ in (23.5)

$$\eta^2 + 1.682\, M\eta - 1.19\, M^2 = 0 \qquad (24.5)$$

which gives $\eta = .535\, M$ (25.5)

But $\eta = \frac{3(1 - K_o)}{(1 + 2K_o)}$ and $M = \frac{6 \times \sin\bar{\phi}}{3 - \sin\bar{\phi}}$

$$\text{Hence } K_o = \frac{1 - .69\sin\bar{\phi}}{1 + .38\sin\bar{\phi}} \qquad (26.5)$$

The prediction of K_o values, for various $\bar{\phi}$ by the equation (26.5) was shown in fig. 18. The predictions by other models are also given for the comparison. The predictions by proposed model fits well with the experimental points for the range of $\bar{\phi}$ values below 30^o.

5.6 STATE BOUNDARY SURFACE

The concept of unique state boundary surface first introduced by Roscoe, Schofield and Wroth in 1958, has been considerably extended and refined in the subsequent works

at Cambridge. Other workers e.g.Henkel and Sowa (1963). Khera and Krizek (1967) have reported results which do not seem to accord with the basic posulate but very little detailed experimental investigations directed specifically towards a critical evaluation of its validity has been reported.

The dependence of λ on the value of η in drained tests were discused in the earlier chapter. This introduces an added complication. The experimental results presented here in Figs. 14, 15 and 17 demonstrate that the state boundary surface for drained test does not agree with the state boundary surface for undrained tests.

The model presented by Roscoe and Burland (1968) predicts the undrained state boundary surface well, but not the drained test results as shown in figs. 15 and 17. It will be seen (Fig. 15 and 17) that the results of all the drained tests lie between the state boundary surfaces obtained for P and q constant tests.

These observations are consistent with the work of Lewin (1971, 1973 and 1975), Newland (1973) and Mathur (1976). As advocated by Lewin, Newland and Mathur, it would appear that a semi-empirical approach similar to the one discussed here to the study of path dependent stress-strain laws for normally consolidated clays may provide an alternative simple and sound method at present time (1976).

CONCLUSIONS

Based on the above study it is evident that

(1) The compression indices λ for various anisotropic consolidation testsare not the same. It decreases with the increase in η .

(2) The proposed model by Mathur (1976) predicts the Stress - Deformation behaviour, anisotropic consolidation behaviour and the coefficient of lateral earth pressure K_o satisfactorily.

Stress path dependent soil behaviour can be adequately predicted by the proposed model. Parameters involved in the proposed stress strain model can be easily obtained with the help of suggested plots. These parameters are constant through out the stress range (up to failure) which is of practical interest in the study of in-situ soil behaviour.

(3) The state boundary surface from drained tests does not agree with the state boundary surface from undrained tests. All the points from drained tests fall in the space enclosed by q-constant tests and p'-constant tests.

(4) Well defined yield point has been shown to exist for p'-constant, q-constant and 'A' stress paths during shear for anisotropically consolidated clay (fig. 10). This provides further confirmation of the existence of well defined yield locus for soft clays as suggested by Mitchell(1970).

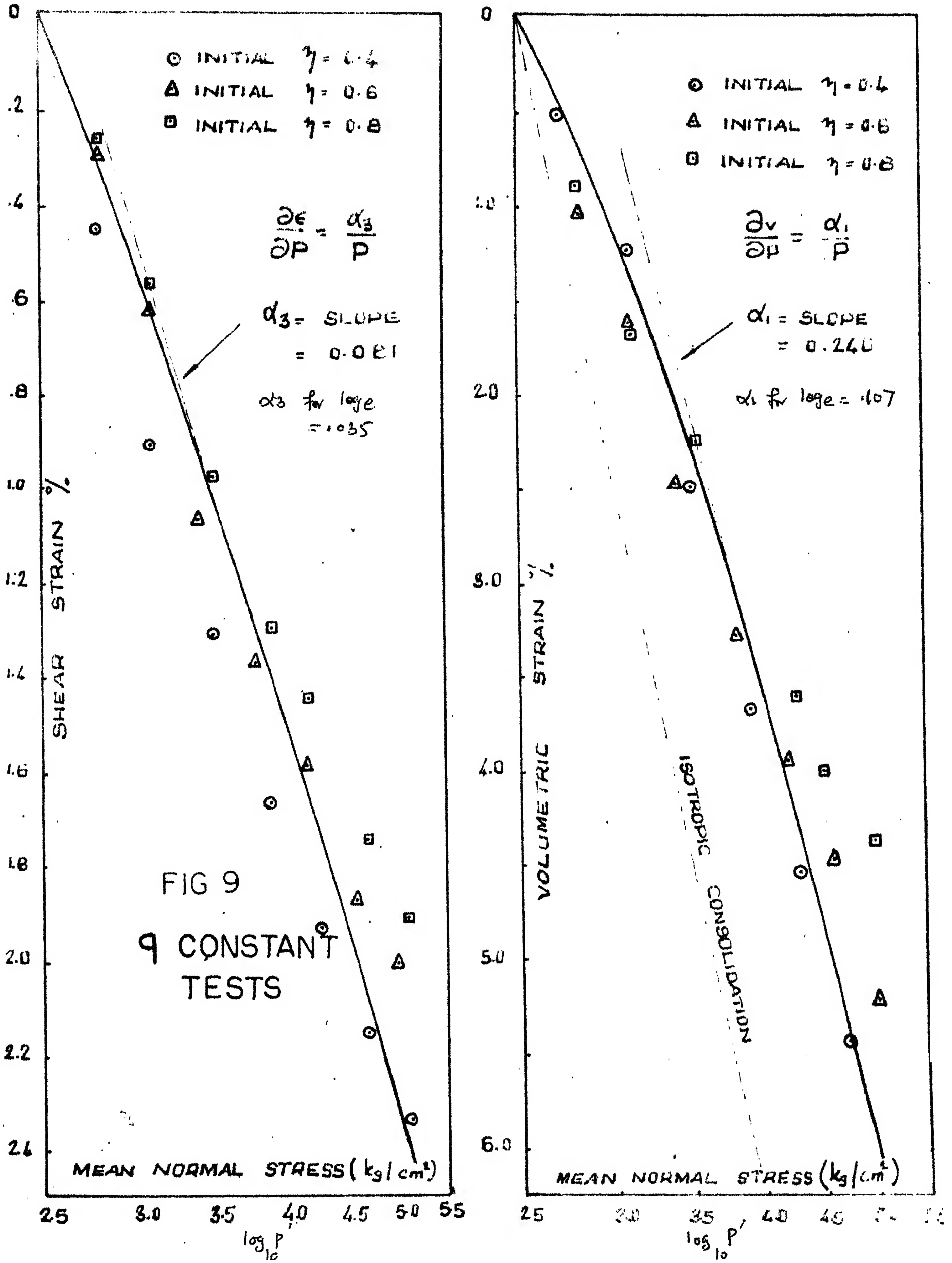
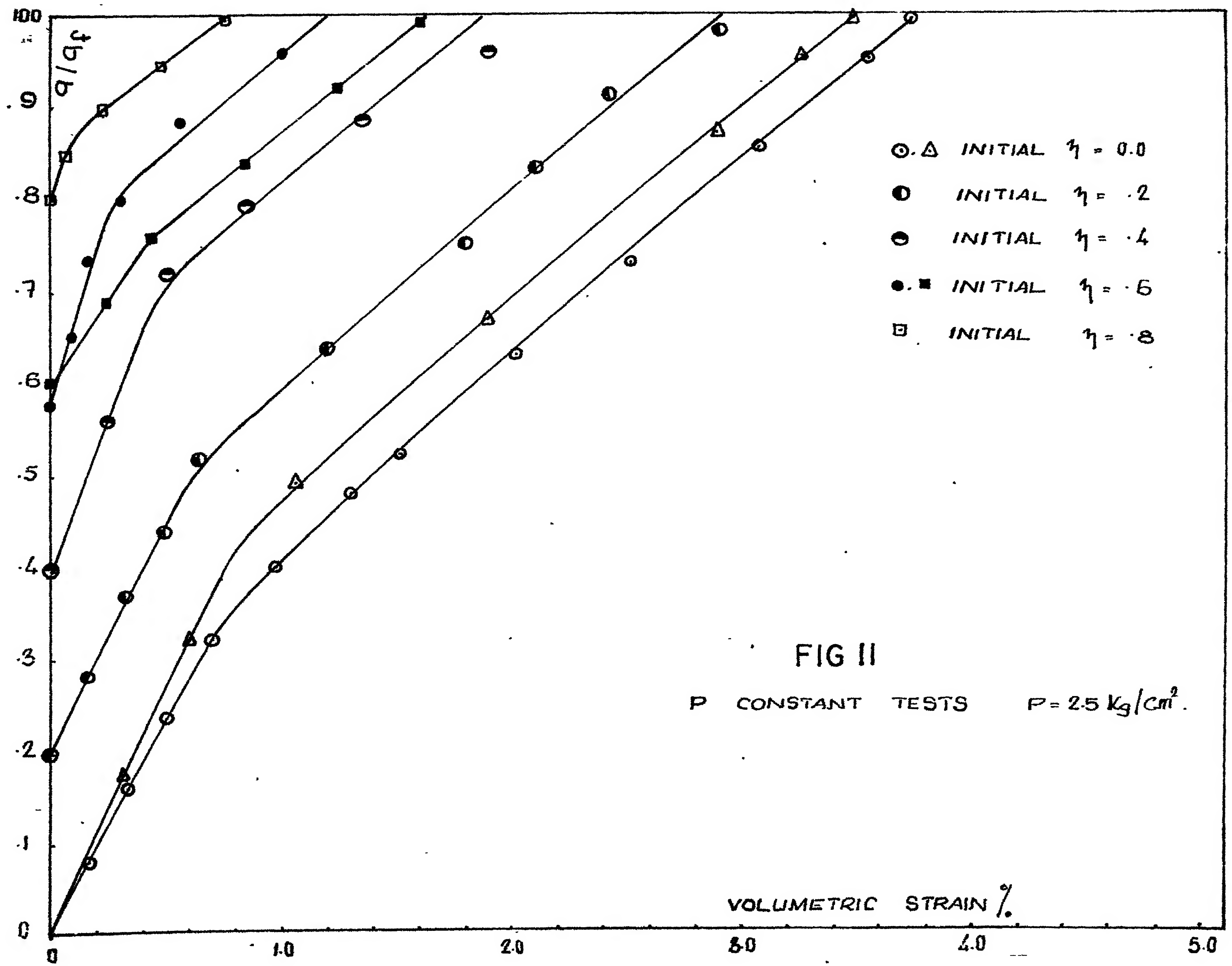


FIG 9

q CONSTANT TESTS


q/qf
100
.9
.8
.7
.6
.5
.4
.3
.2
.1
0
0
1.0
2.0
3.0
4.0
5.0
VOLUMETRIC STRAIN %
⊙.△ INITIAL η = 0.0
◐ INITIAL η = ·2
◒ INITIAL η = ·4
●.■ INITIAL η = ·6
⊡ INITIAL η = ·8
FIG II
P CONSTANT TESTS P = 2.5 Kg/cm²

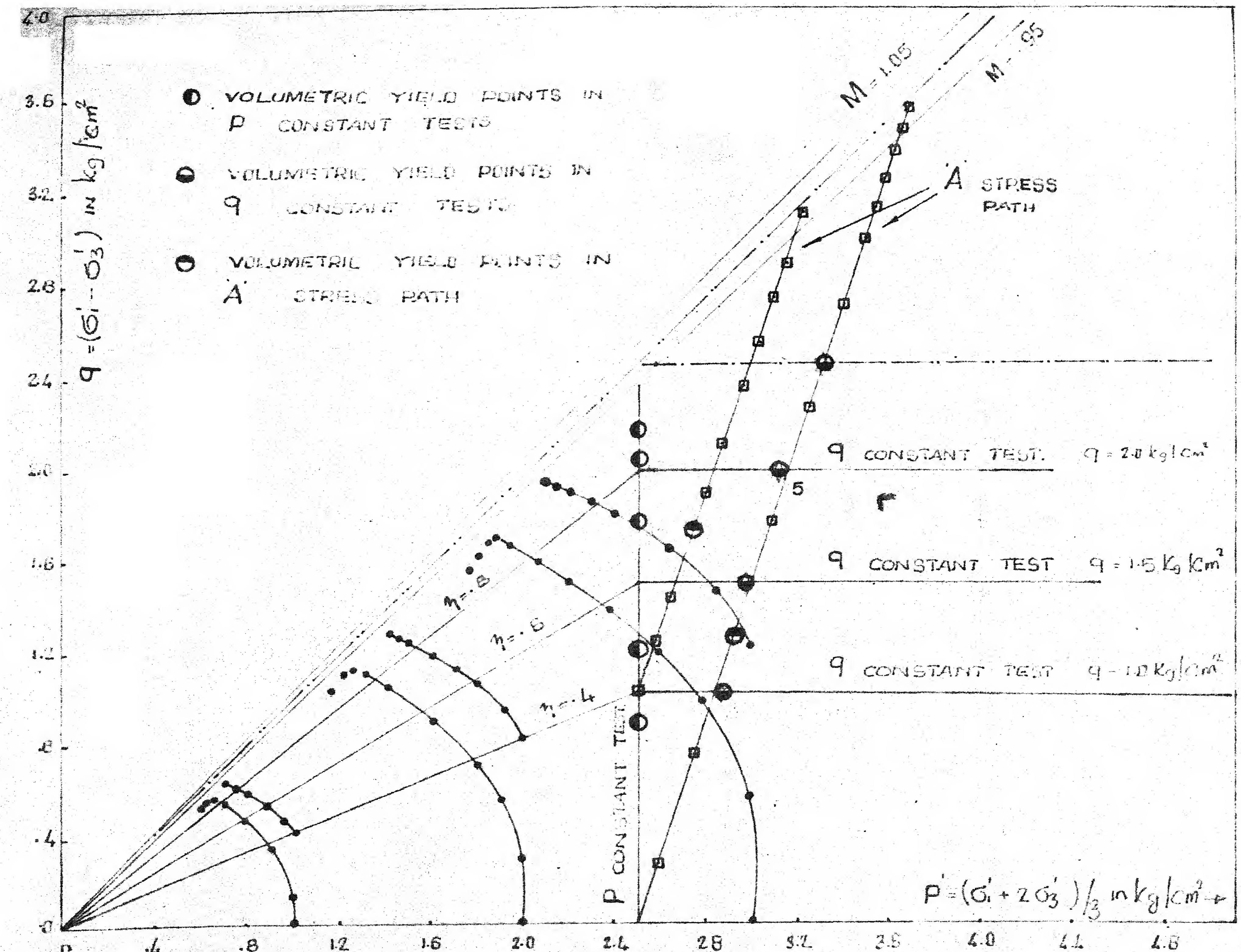


FIG.10 STRESS PATHS & YIELD POINTS

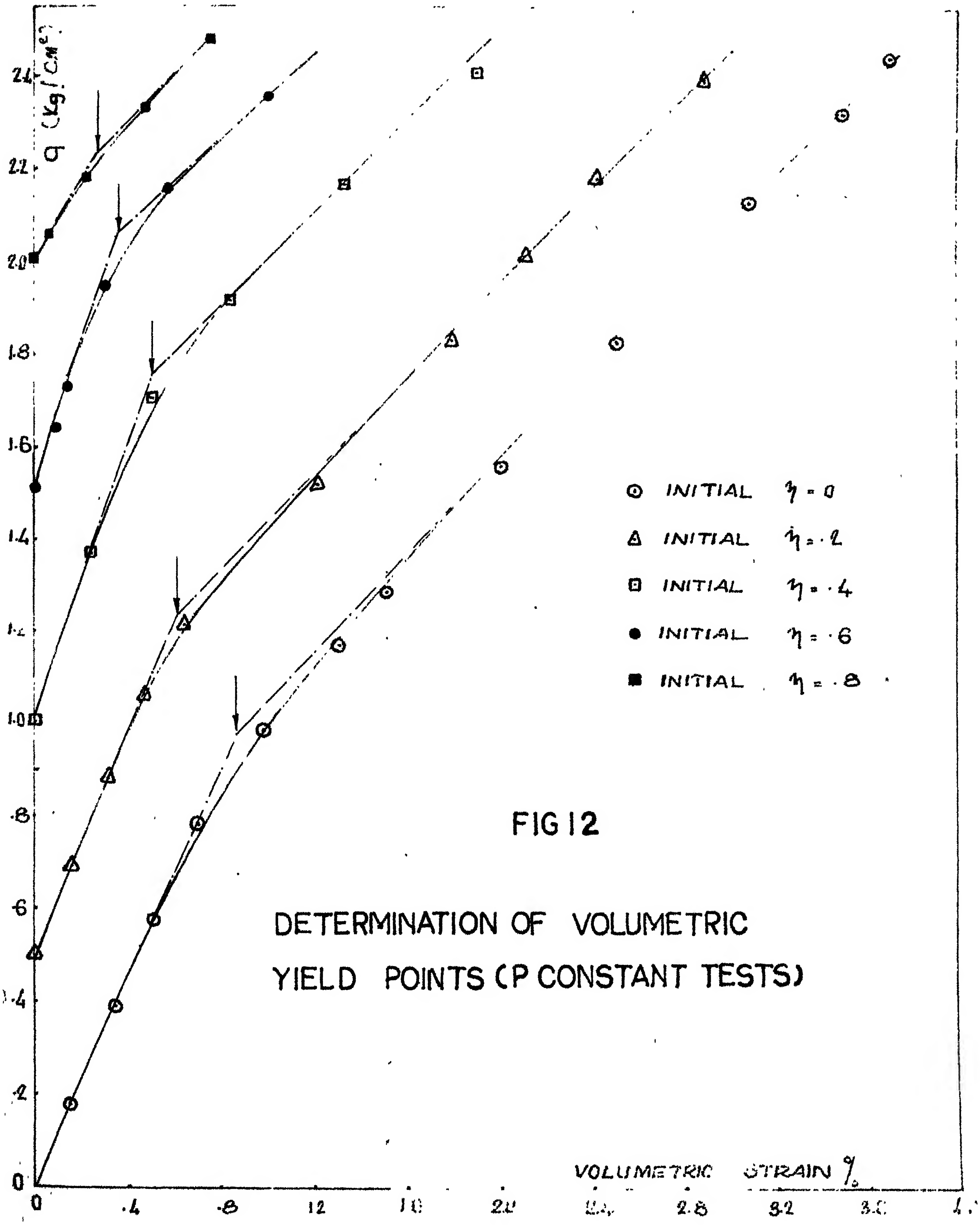


FIG 12

DETERMINATION OF VOLUMETRIC
YIELD POINTS (P CONSTANT TESTS)

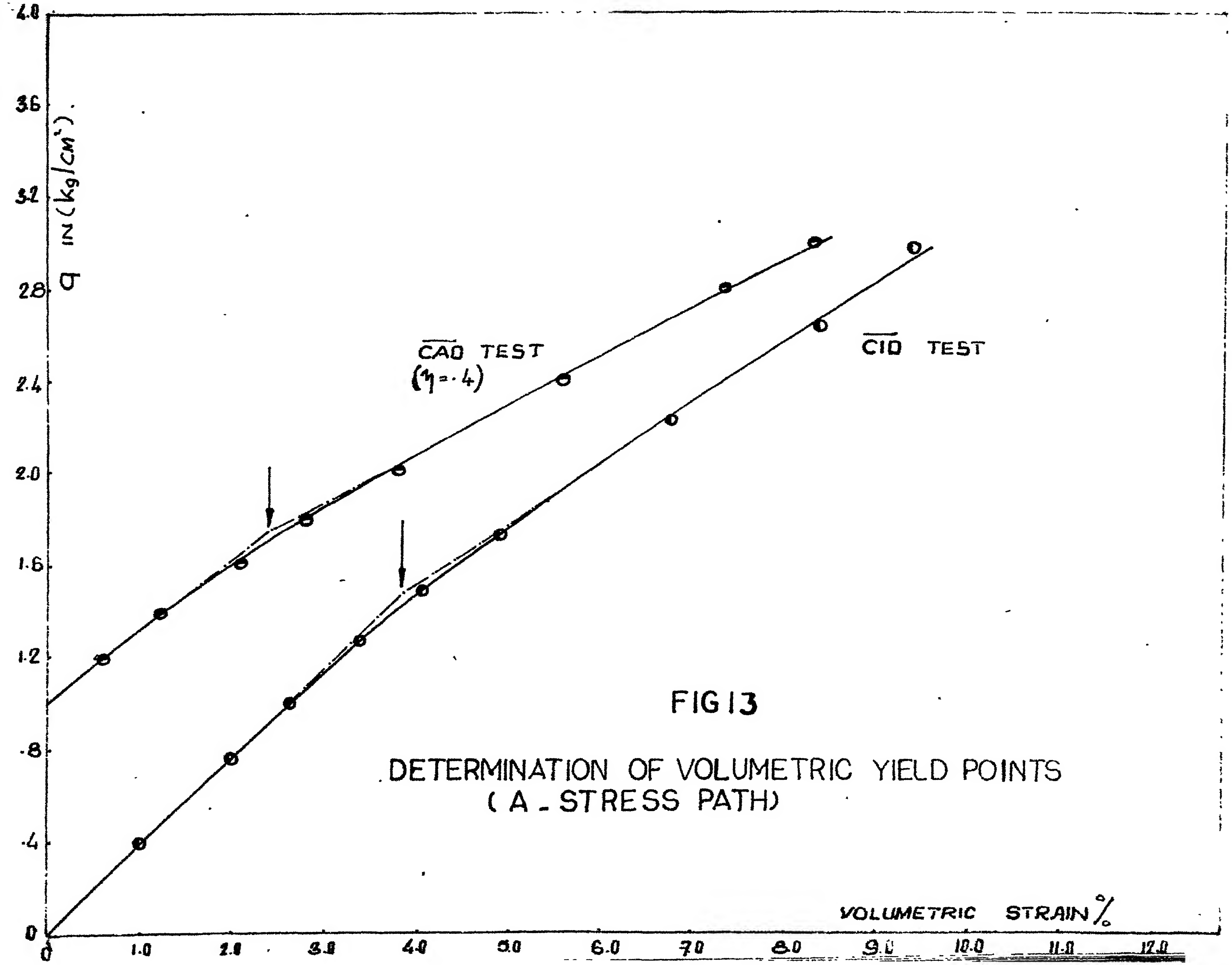


FIG 13

DETERMINATION OF VOLUMETRIC YIELD POINTS
(A - STRESS PATH)

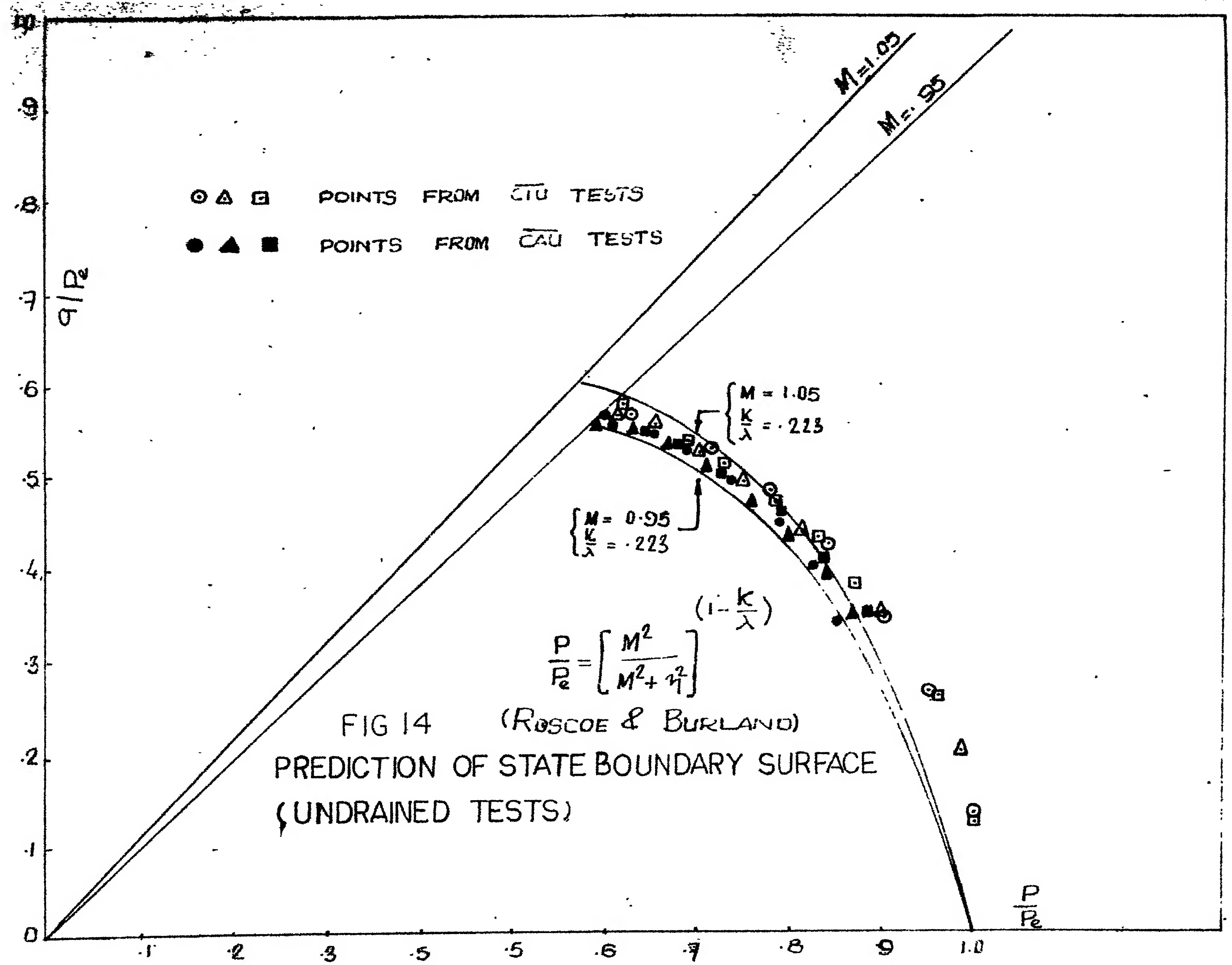


FIG 14 (ROSCOE & BURLAND)
PREDICTION OF STATE BOUNDARY SURFACE
(UNDRAINED TESTS)

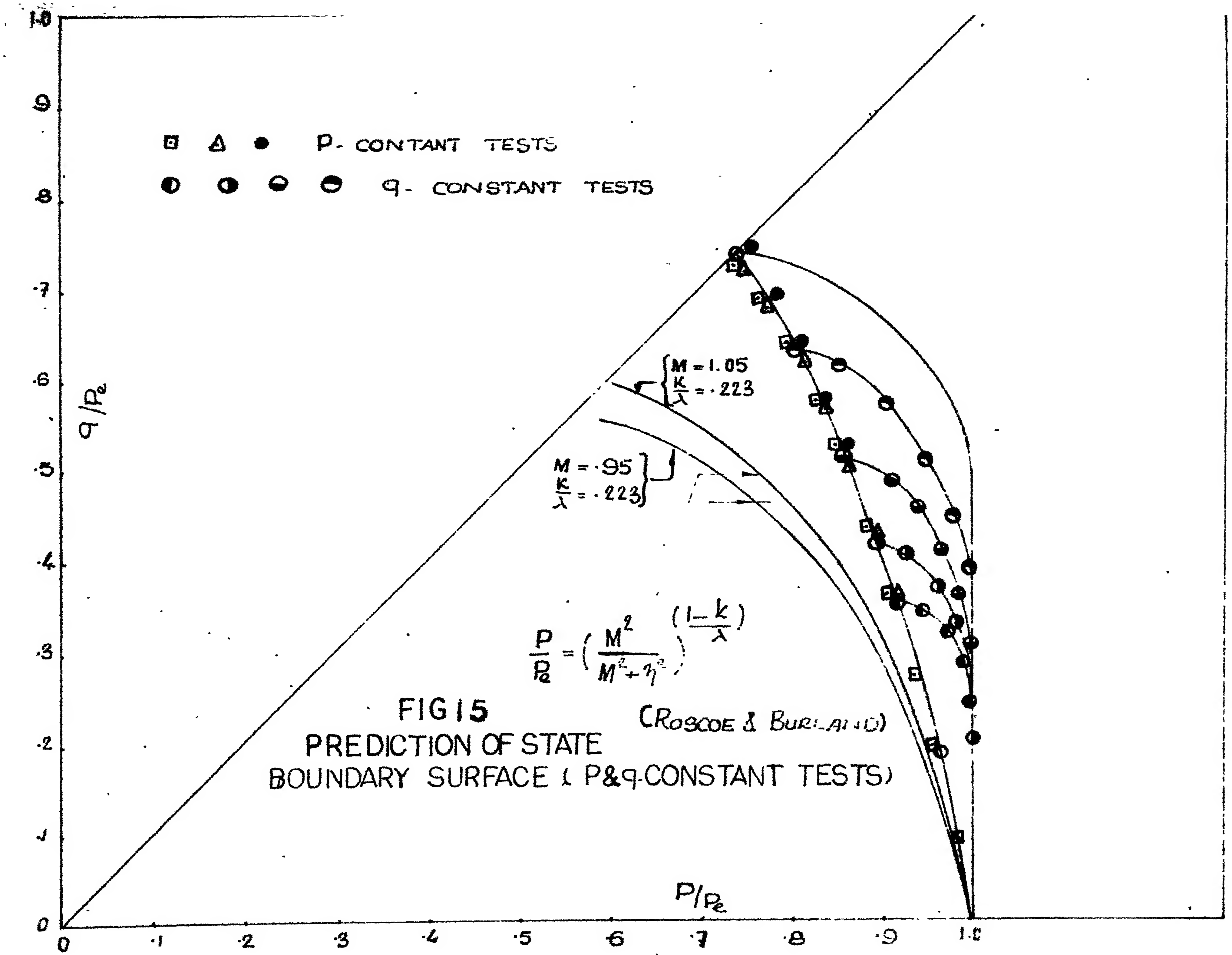


FIG 15
PREDICTION OF STATE
BOUNDARY SURFACE (P&q-CONSTANT TESTS)

FIG 16 PREDICTION OF FLOW RULE

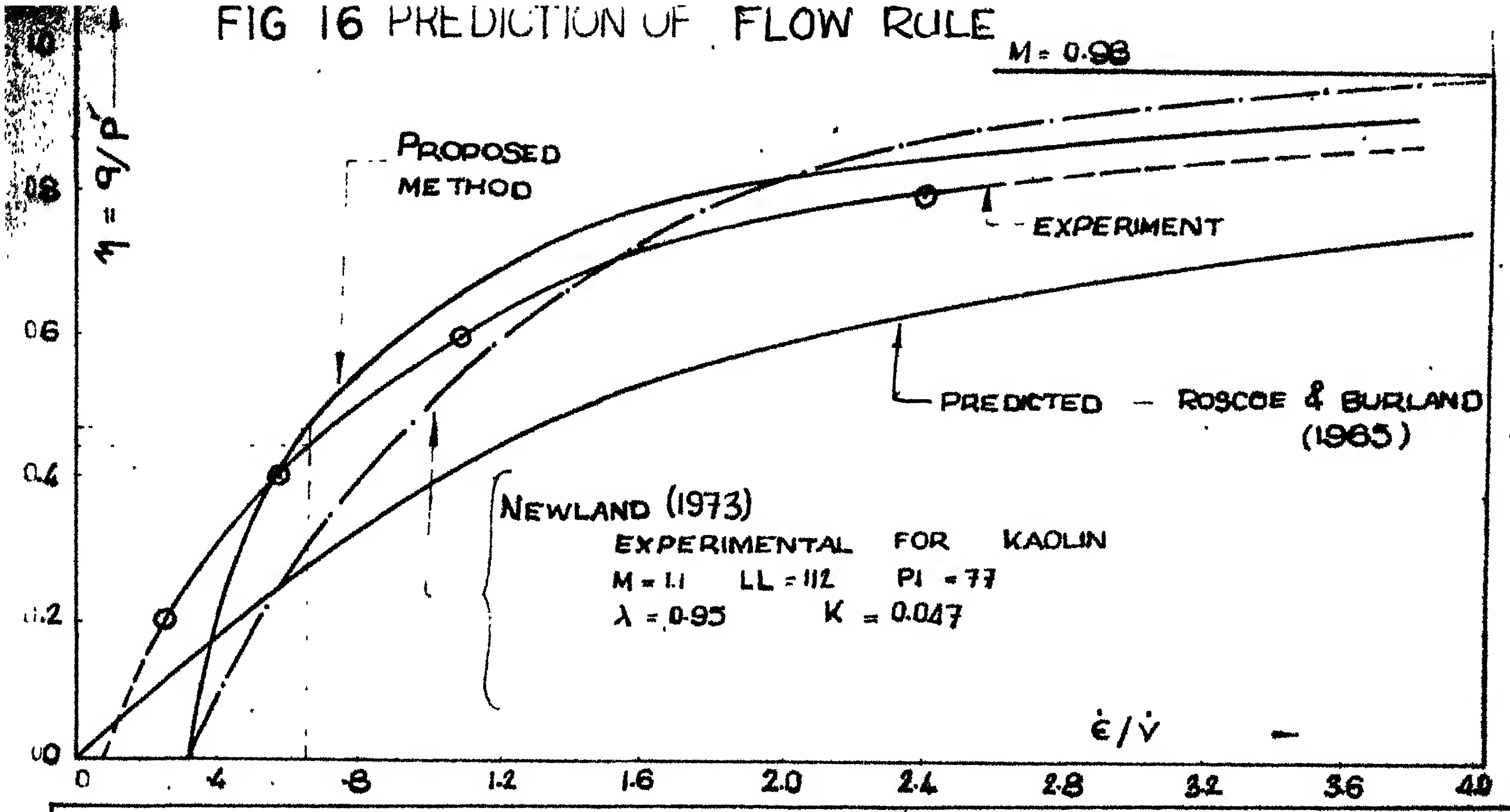
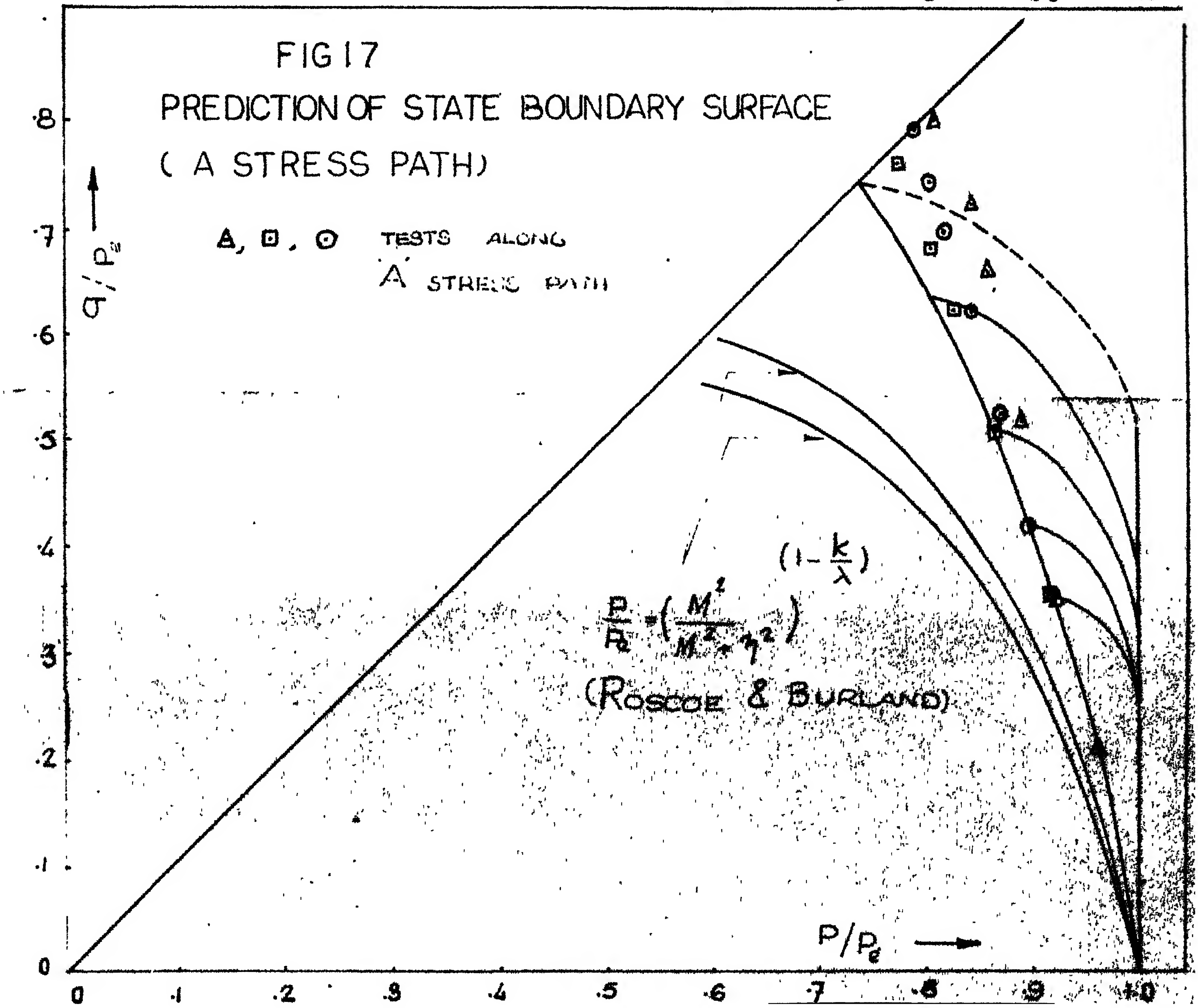
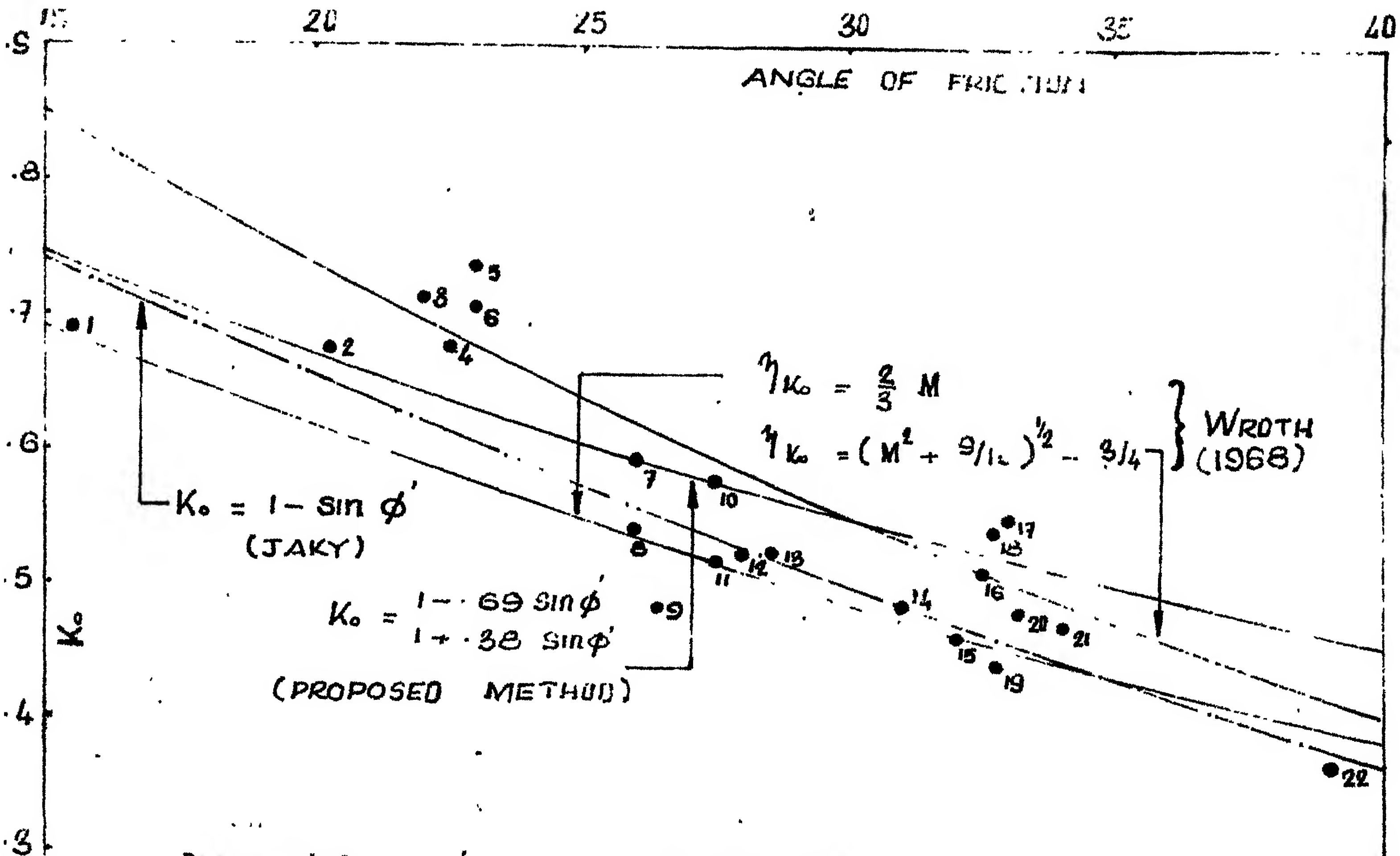


FIG 17

PREDICTION OF STATE BOUNDARY SURFACE

(A STRESS PATH)





1	Bearpaw shale	Brooker (1964)
2.	Kaolin	Bishop (1958)
3	Weald clay	"
4	London clay	Brooker (1964)
5	Gault clay	Thomson (1962)
6	Kaolin	Burland (1967)
7	Weald clay	Sowa (1963)
8	Weald clay	Brooker (1964)
9	Chicago clay	" "
10	Yarmouth clay	Bishop (1968)
11	Goose later flour	Brooker (1964)
12	Kristransand clay	Simons (1958)
13	New field clay	Namy (1970)
14	Oslo marine clay	Simons (1958)
15	Leighton Buzyard sand	Cole (1967)
16	Boston clay	Ladd (1965)
17	Sand	Simons (1958)
18	Belgian sand	Wade (1963)
19.	Chow stoke clay	Bishop (1958)
20	Scott clay	Ladanyi et al (1965)
21	Sand (loose)	Bishop (1958)
22	Sand (dense)	Bishop (1958)

FIG18 PREDICTION OF K_o

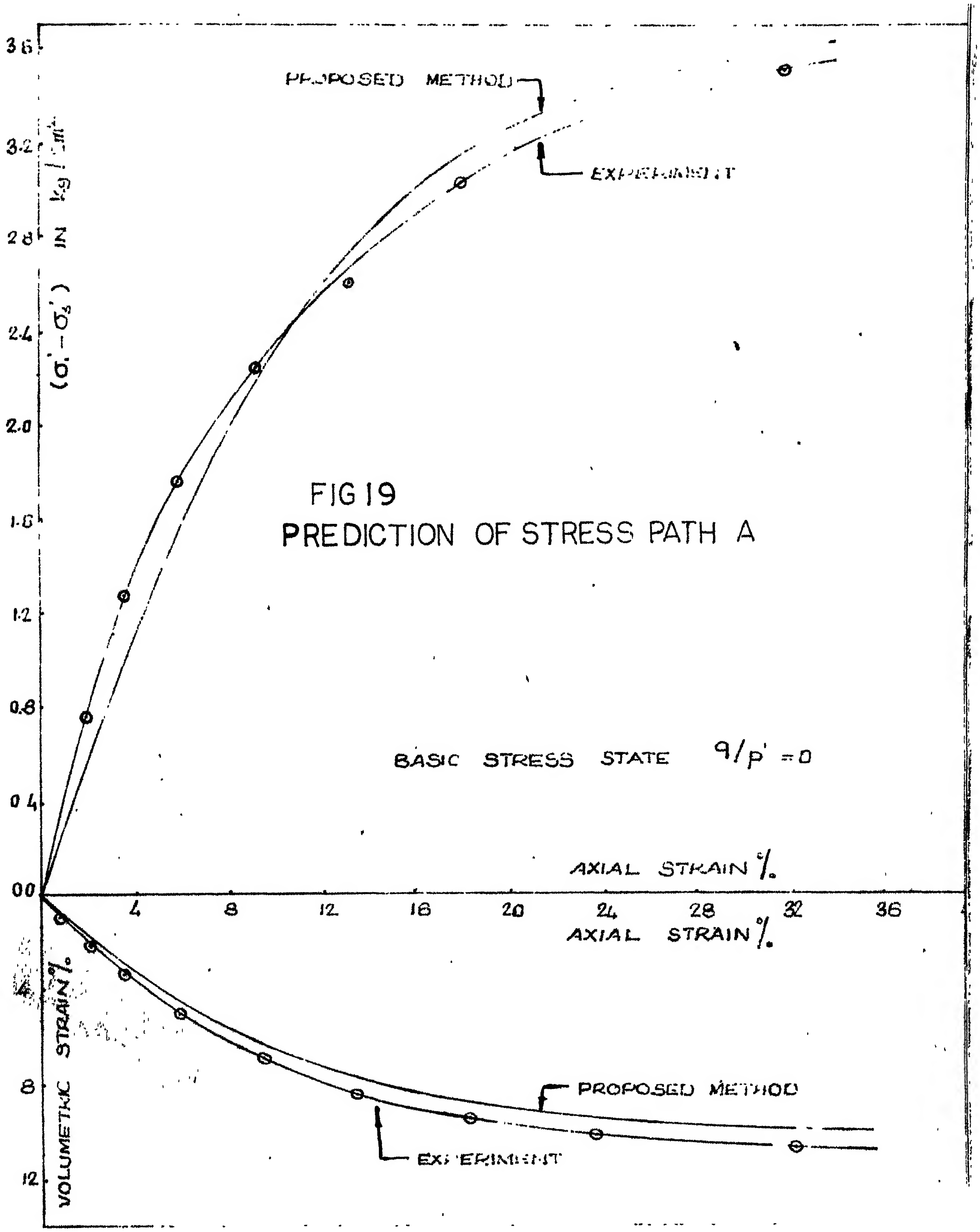


FIG 19
PREDICTION OF STRESS PATH A

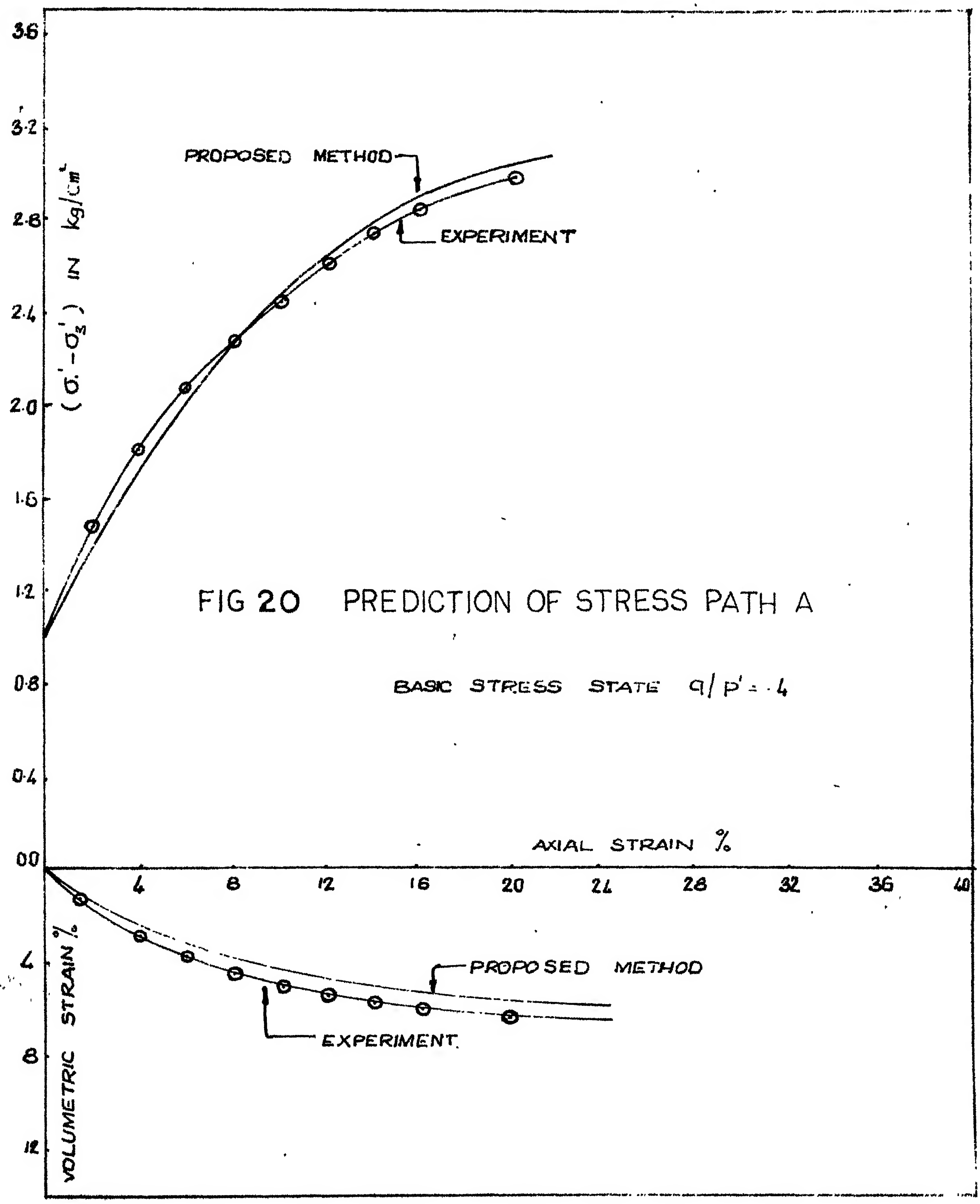


FIG 20 PREDICTION OF STRESS PATH A

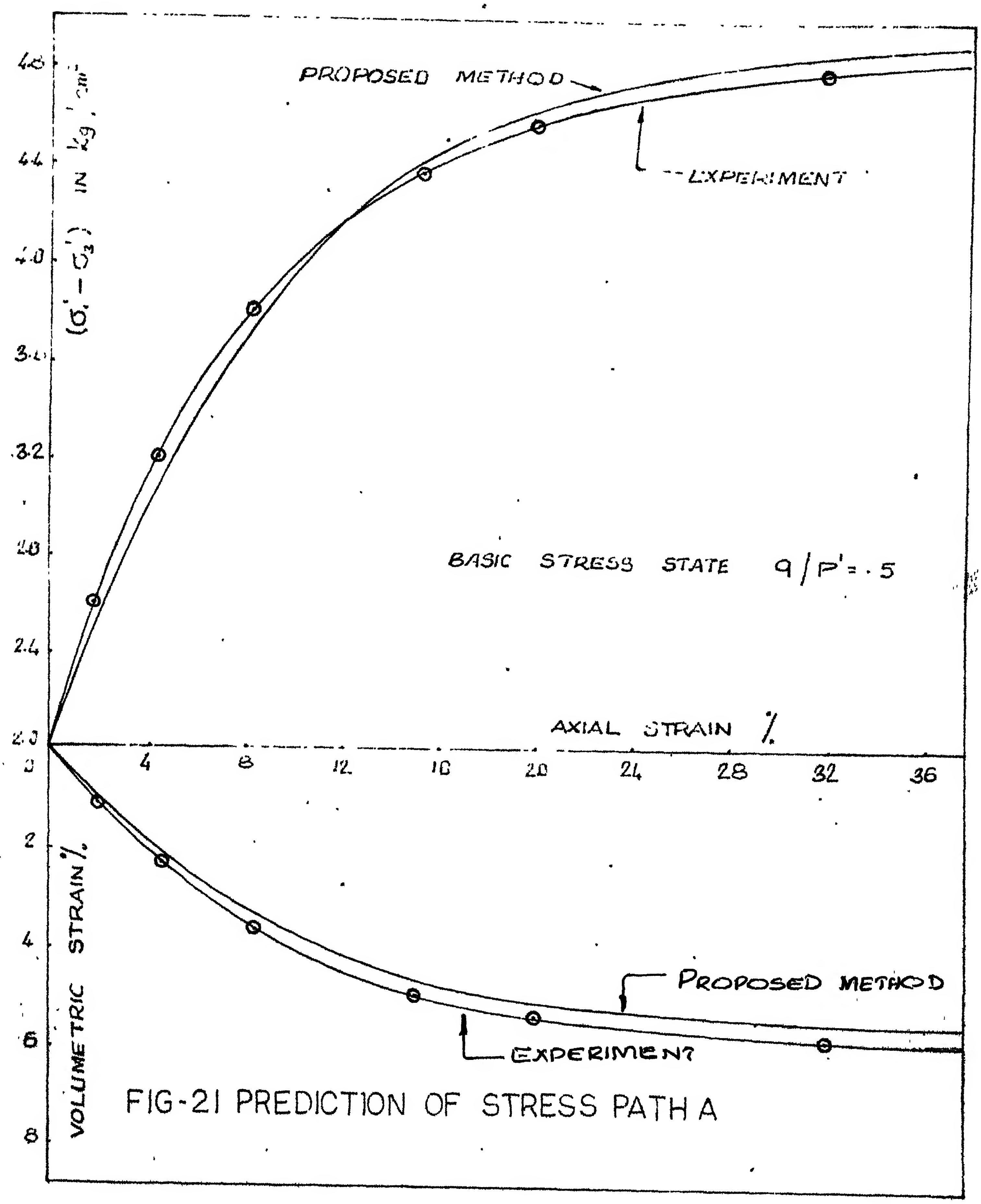


FIG-21 PREDICTION OF STRESS PATH A

BIBLIOGRAPHY

Bishop, A.W., and Henkel, D.J. (1962), 'The measurement of soil properties in the Triaxial Test', Edward Arnold (Publishers) Ltd.

Bishop, A.W. (1972), 'Shear strength parameters for undisturbed and remoulded soil properties', Proc. of the Roscoe Memorial Symposium, Cambridge University, Foulis and Co. Ltd., March.

Burland, J.B. (1965), 'The yielding and Dilation of clay', Correspondence, Geotechnique, Vol. 15, No. 2, pp. 211-214.

Calladine, C.R. (1971), 'A Microstructural view of the Mechanical Properties of Saturated Clay', Geotechnique, December, pp. 391-415.

Calladine, C.R. (1973), 'Overconsolidated Clay: A Microstructural view'. Proc. of Symposium on Plasticity in Soil Mechanics, Cambridge, pp. 144-158.

Henkel, D.J. (1972), 'The Relevance of Laboratory Measured parameters in Field studies', Proc. Roscoe Memorial Symposium, Foulis, pp. 669-675.

Henkel, D.J. (1958), 'The Relationship between the strength pore water pressure and volume change characteristics of saturated clays', Geotechnique, Vol. 9, pp. 119-135.

Henkel, D.J. (1960), 'The Relationship between the Effective stresses and water content in the saturated clays', Geotechnique, Vol. 10, pp. 41-54.

Henkel, D.J. and Sowa, V.A. (1963), 'The Influence of stress History on stress paths in undrained triaxial tests on clay', Laboratory shear testing of soils, NRC, Canada, ASTM, pp. 280-291.

Hvorslev,M.J. (1960), 'Physical Components of the shear strength of saturated clays',ASCE Research Conf. on shear strength ofCohesive soils, Boulder, Colorado, pp. 169-273.

Ladd, C.C. (1965), 'Stress strain behaviour of Anisotropically consolidated clays during undrained shear', Proc. VI Int. Conf. Soil Mech. and Found. Engg., Montreal, Vol. 1, pp. 282-286.

Lambe, T.W. (1967), 'Stress Path Method', Journal of Soil Mech. and Found. Division, ASCE, pp. 43-67.

Lambe, T.W. and Whitman, R.V. (1969), 'Soil Mechanics', John Wiley and Sons.

Lambe, T.W. (1973), 'Predictions in Soil Engineering', Thirtenth Rankin Lecture, Geotechnique, Vol. 23, No. 2, pp. 149-202.

Lewin, P.I. and Burland, J.B. (1970), 'Stress probe experiments on saturated normally consolidated clay', Geotechnique, Vol. 20, No. 1, pp. 38-56.

Mathur, S.K. (1976), 'STRESS STRAIN TIME Behaviour of anisotropically consolidated Marine clay under various stress paths'. Ph.D. Thesis, Indian Institute of Technology, Kanpur.

Mitchell, R.J. (1970), 'On the yielding and Mechanical strength of Leda clays', Canadian Geotechnical Journal, Vol. VII, No. 3, pp. 297-312.

Newland, P.L. (1973), 'An experimental study of the stress strain characteristics of a 'Wet' clay and the Relevence to settlement analysis', Division of Applied Geomechanics Technical Report No. 15, CSIRO, Australia.

Poorooshasb, H.B. and Roscoe, K.H. (1963), 'A Graphical approach to the problem of the stress strain relation of normally consolidated clays', Symposium on shear testing of soils, ASTM, STP 361, pp. 258-264.

Prevost, J.H. (1974), 'Soil stress-strain strength models based on plasticity theory', Ph.D. Thesis, Standard University, California.

Prevost, J.H. and Hoeg, K. (1975), 'Effective stress strain strength model for soils', Journal of the Geotechnical Engineering Division, ASCE, Vol. 101, GT3, March, pp. 259-278.

Rendulic, L. (1936), 'Relation between void ratio and principal stresses for a remoulded silty clay', Proc. First Int. Conf. Soil Mech. and Found. Engg., Cambridge, Vol. 3, pp. 48-31.

Roscoe, K.H., Schofield, A.N. and Wroth, C.P. (1958)' on the yielding of soils', Geotechnique, Vol. 8, pp.22-53

Roscoe, K.H., and Poorooshasb, H.B. (1963), ' A theoretical and experimental study of strains in triaxial compression tests on normally consolidated clays', Geotechnique, Vol. 13, No. 1, pp. 12-38.

Roscoe, K.H., Schofied, A.N. and Thurairajah, A. (1963), 'Yielding of clays in states wetter than critical', Geotechnique, Vol. 13, No. 3, pp. 211-240.

Roscoe, K.H. and Schofield, A.N. (1963), 'Mechanical behaviour of an idealized 'wet clay' . P roc. 11 European Conf. Soil. Mech., Wiesbaden.

Roscoe, K.H. and Thurairajah, A. (1964), 'On the uniqueness of yield surfaces for wet clays 'IUTAM symposium on Rheology and Soil Mech. Grenoble, pp. 364-384.

Roscoe, K.H. and Burland, J.B. (1968), 'On the Generalized stress strain behaviour of 'wet' clay', symp. on Engineering Plasticity, Cambridge University Press, pp. 535-609.

Roscoe, K.H. (1970), 'The Influence of strains in soil mechanics, Geotechnique, Vol. 20, No. 2, pp. 129-170.

Schofield, A.N. and Wroth, C.P. (1968), 'Critical state soil Mechanics' McGraw Hill England.